AD 727892

FTD-MT-24-349-70

FOREIGN TECHNOLOGY DIVISION

MINING STRUCTURES

(Selected Chapters)

by

S. G. Matveyev

Approved for public release; distribution unlimited.

406

UNCLASSIFIED

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author) Foreign Technology Division Air Force Systems Command U. S. Air Force	2a. REPORT SECURITY CLASSIFICATION UNCLASSIFIED 2b. GROUP

3. REPORT TITLE

MINING STRUCTURES
(Selected Chapters)

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)

Translation

5. AUTHOR(S) (First name, middle initial, last name)

Matveyev, S. G.

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
1962	397	

8a. CONTRACT OR GRANT NO. b. PROJECT NO. 6010201 c. d. DIA Task No. T66-01-8A	9a. ORIGINATOR'S REPORT NUMBER(S) FTD-MT-24-349-70 9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)

10. DISTRIBUTION STATEMENT

Approved for public release; distribution unlimited.

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	Foreign Technology Division Wright-Patterson AFB, Ohio

13. ABSTRACT

This book deals mainly with the detailed design of mine equipment and structures, chiefly pile drivers multiple-rope lifts, deckhouses, loading and unloading construction, conveyor galleries, hoppers and storage facilities. The author discusses the various designs (and even slight modifications) of the mining equipment, using metal and non-metal materials. Methods are given to maintain the equipment under various and sundry conditions of operation. The author elaborates to a considerable degree on pre-stressed and reinforced concrete structures as applied to mining construction. Much attention is also given to designinig of structures, seismically, because of rock blasting in the immediate area of the structures.

DD FORM 1 NOV 65 1473

UNCLASSIFIED

Security Classification

KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Mining Machinery						
Mining Geology						
Reinforced Concrete						

FTD-MT-24-349-70

EDITED MACHINE TRANSLATION

MINING STRUCTURES (Selected Chapters)

By: S. G. Matveyev

English pages: 397

Source: Rudnichnyye Sooruzheniya, 1962, Moscow, Izd-vo Literatury po Gornomu Delu, pp. 46-123, 202-325, and 451-557.

This document is a SYSTRAN machine aided translation, post-edited for technical accuracy by: Ray E. Zarza.

UR/0000/62/000/000

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U. S. BOARD ON GEOGRAPHIC NAMES TRANSLITERATION SYSTEM

Block	Italic	Transliteration	Block	Italic	Transliteration
А а	*А а*	A, a	Р р	*Р р*	R, r
Б б	*Б б*	B, b	С с	*С с*	S, s
В в	*В в*	V, v	Т т	*Т т*	T, t
Г г	*Г г*	G, g	У у	*У у*	U, u
Д д	*Д д*	D, d	Ф ф	*Ф ф*	F, f
Е е	*Е е*	Ye, ye; E, e*	Х х	*Х х*	Kh, kh
Ж ж	*Ж ж*	Zh, zh	Ц ц	*Ц ц*	Ts, ts
З з	*З з*	Z, z	Ч ч	*Ч ч*	Ch, ch
И и	*И и*	I, i	Ш ш	*Ш ш*	Sh, sh
Й й	*Й й*	Y, y	Щ щ	*Щ щ*	Shch, shch
К к	*К к*	K, k	Ъ ъ	*Ъ ъ*	"
Л л	*Л л*	L, l	Ы ы	*Ы ы*	Y, y
М м	*М м*	M, m	Ь ь	*Ь ь*	'
Н н	*Н н*	N, n	Э э	*Э э*	E, e
О о	*О о*	O, o	Ю ю	*Ю ю*	Yu, yu
П п	*П п*	P, p	Я я	*Я я*	Ya, ya

* ye initially, after vowels, and after ъ, ь; e elsewhere. When written as ё in Russian, transliterate as yё or ё. The use of diacritical marks is preferred, but such marks may be omitted when expediency dictates.

FOLLOWING ARE THE CORRESPONDING RUSSIAN AND ENGLISH DESIGNATIONS OF THE TRIGONOMETRIC FUNCTIONS

Russian	English
sin	sin
cos	cos
tg	tan
ctg	cot
sec	sec
cosec	csc
sh	sinh
ch	cosh
th	tanh
cth	coth
sch	sech
csch	csch
arc sin	$\sin^{-1}$
arc cos	$\cos^{-1}$
arc tg	$\tan^{-1}$
arc ctg	$\cot^{-1}$
arc sec	$\sec^{-1}$
arc cosec	$\csc^{-1}$
arc sh	$\sinh^{-1}$
arc ch	$\cosh^{-1}$
arc th	$\tanh^{-1}$
arc cth	$\coth^{-1}$
arc sch	sech^{-1}
arc csch	csch^{-1}
rot	curl
lg	log

Translator's note: On several occasions, symbols found in formulae and calculations appear to have been rendered incorrectly in the original document. They will be shown exactly as they appear in the original.

CHAPTER II

THE SPECIFIC CONDITIONS OF THE DESIGN, EXCAVATION AND OPERATION OF MINE STRUCTURES IN THE MINING INDUSTRY

1. General Information

During the operation of mines and open pits in the mining industry as a result of using mass explosive work, the undermining of sections of the surface by mining operations, the uneven deposition of artificially created earth fills at the mines, etc., all produce specific conditions, which unfavorably affect the commercial state of the mine structure.

On the basis of observation and research under these conditions during designing and excavation of the structure special measures are taken for their protection and for the normal operating conditions by means of strengthening the designs, the utilization of special structural solutions, and others.

2. The Protection of the Structures in the Areas of Blasting

In many mining enterprises (especially in the open pit mines) one simultaneously detonates a large quantity of explosives.

As a result of blasting vibrations of the bedrock and soil, analogous to seismic vibrations during earthquakes appear.

During the blasts the virbrations attenuate rather rapidly (for 1-10 s) the greatest danger for buildings and structures are the vibrations having a span of 0.1-0.5 s. The large values of the periods of vibration are characteristic for soils, which possess low bearing capacity, and irrigated soils. Many mine structures and buildings have a natural period of vibrations within the limits of 0.2-0.5 s, close to the period of seismic vibrations, characteristic for slightly water saturated soils. In such soils the greatest danger of failure of the structure can arise. In stony soils, the period vibrations usually does not exceed 0.1 s, and the probability of the emergence of the phenomena of resonance as well as failure is considerably reduced. According to the data of observations it has been established, that with single blasts and at velocities of vibrations, less than 0.1-0.14 m/s, the failures of the structures are not observed.

According to the Rules of Safety during blasting the formula accepted in order to determine seismically the extent of a normal (with a funnel of a normal ejection) radius of the danger zone

$$r_c^n = K_c \sqrt[3]{q},$$

where K_c – the coefficient, depending on the physical-mechanical features of the soils; q – the weight of the charge, kg.

The values of the radii of the seismically danger zones, determined according to the given formula for the charges of a different weight and for various conditions, are given in under Table 2.

Table 2.

The soils as foundation of protected structures		Approximate value of the coefficient K_c	Weight of the charge, t				
			1	10	50	100	200
Characteristics	Name		Radii of seismically dangerous zones, m				
Weak soils	Water saturated (quick ground and peat)	20	200	430	740	930	1100
	Fill and natural soil material	15	150	320	550	700	880
Soils of the average bearing capacity, clayey, stony	Clayey	9	90	195	330	420	475
	Sandy	8	80	170	300	370	420
	Gravely and stony	7	70	159	260	330	370
Rocky soils	Disentegrated bedrock	5	50	110	185	230	290
	Dense bedrock	3	30	65	110	140	175

With the funnels of the reduced and amplified ejection it is necessary to multiply the accepted values of r_c^H in Table 2 by the coefficient α, depending on the index of the action of the explosion n (the relationship of the radius of the funnel of explosion to the line of least resistance).

For funnels of normal ejection $n = 1$; $\alpha = 1$

For funnels of reduced ejection $n = 0.5$; $\alpha = 1.2$

For funnels of amplified ejection $n = 2-3$; $\alpha = 0.8-0.7$

In general the value of the radius is seismically the danger zone during the explosions

$$r_c = \alpha K_c \sqrt[3]{q}.$$

During explosions in water saturated soils and in water, the value of the coefficient K_c needs to be increased by 1.5-2 times.

During mass blasting the distance of the industrical grounds from the blasting site should be determined, based on the following conditions.

1. People, who are located on the industrial grounds, should not be subjected to danger as the result of the dispersion of fragments of the blasted rocks.

2. The buildings and structures should be located beyond the limits of a seismically dangerous zone.

Inasmuch as the radius of the danger zone according to the dispersion of the fragments of the blasted rock and in accordance with the Rules of Safety, is applied within the limits of 200-400 m, and the normal radii are seismically safe zones in most cases at less than 400 m, the impression is created that the reference of an industrial site beyond the limits of the danger zone based on a calamity is also guaranteed for the seismic safety of buildings and structures. Practical observations have indicated that as a result of the repeated frequency of explosions in constructions, located outside the limits of the radius of the seismically dangerous zone, numerous criteria of the destructive pressure of the explosions in the form of cracks in the walls, opening of seams in overlaps, floors, partitions, and others are frequently observed. These phenomena, frequently imperceptible during the first explosions, manifest themselves with the repetition and intensification of the latter, and they can be clearly expressed during a further continuation and development of the blasting. With the realization of the complex of measures for the protection of buildings and structures in the such cases from the effect of seismic vibrations by a force of approximately seven on the Ball scale the destructive pressure of the explosion usually does not manifest itself.

The emergence of cracks in the walls of conventional stone buildings with the simultaneous lack of damage in buildings with

antiseismic designs accepted in the USSR, the scale of tremors corresponds to the number seven reading on the Ball scale of seismicity (GOST 6249-52).

The distribution of the seismic phenomena during systemically repeated explosions beyond the limits of the radius of the seismically dangerous zone (determined during single explosions) can be explained as follows.

According to the data in the practice of blasting using systematically repeated explosions, the velocities of the vibrations of the structures should be reduced to 2-3 cm/s. In individual cases during short periods of operation, and with the possibility of also presuming the occurrence of insignificant cracks and the flaking of plaster, the velocity values of vibrations can constitute 4-6 cm/s.

According to the investigations of M. A. Sadovskiy, the decrease in the velocities of vibrations to 4-6 cm/s is assured by the spacing of buildings from the blasting site by a distance of approximately 1.4-2 times in comparison with that determined according to a formula of the radii of seismically dangerous zones (r_c). The further decrease in the velocities of vibrations to 2-3 cm/s is attained by an increase in the distances by 2-4 times.

An increase in the distance between an industrial site or a single standing building, and the blasting place of systematically repeated explosions by a three-fold radius of the seismically dangerous zone derived from the formula for a single explosion, is a sufficient guarantee of the seismic reliability of buildings and structures. If, based on any considerations whatsoever the placement of an industrial site beyond the limits outlined the three-fold radius increase of the seismically dangerous zone should be infeasible or inexpedient, it is recommended in the design and construction of structures to carry out a complex of antiseismic measures, used in the region of Ball scale seven seismicity with several of the changes

recommended below. In this instance the seismic safety of the buildings will also be taken care of.

If the mine struture is placed within the limits of a two-fold radius increase of a seismically dangerous zone, the awareness of the complex of antiseismic measures should be played up, and all the bearing elements of the structure should be calculated for seismic loads.

In exceptional cases, when for individual buildings and structures it is necessary to locate them within the limits of a seismically dangerous zone (for the calculation for single explosions), an awareness of the measures correponding to Ball scale seven should be provided for seismicity.

The approximate dependence between seismicity, expressed in the Ball scale, and the distance of the explosions to the structures and buildings is shown in Fig. 35. A distance to the blast site is expressed here in radii of the seismically dangerous zones r_c. Seismicity also in this case is characterized by the criteria of failure and of damage to the building structure (GOST 6249-52) as a result of systematically repeated explosions.

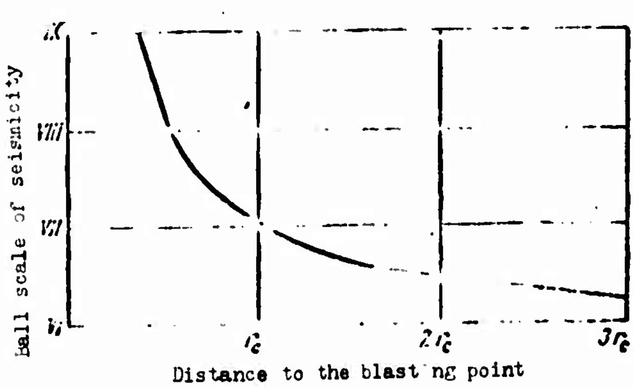

Fig. 35. Dependence between a distance to the blasting point (expressed in values of r_c) and the Ball scale number seismicity.

The calculation of seismic loads is conducted under the assumption of the static action of seismic forces, whose distribution is assumed to depend on the location of the masses in the construction.

These forces can have any direction but predominantly they are assumed to act horizontally. In the calculation of couplings, anchor bolts, bracings of supports, one takes into account the seismic forces that shear or stretch these connections.

The seismic calculation of such a mine structure, as surface buildings, haul and loading docks and tunnels, conveyer tunnels, bridges, passages, support and temporary structures for stock piles and retaining walls can be done according to standards for structures in seismic regions, and the seismic loads can be found from the expression

$$S - QK_c\alpha,$$

where Q – vertical load (the weight of the structural elements themselves, soil, temporary loads); K_c – the seismic coefficient, equal at a seismicity of Ball scale reading of 7, equal to 1/40; α – the coefficient, assumed; for high supports of bridges (at a ratio of the altitude to the width – more than five), high retaining walls, towers, columns $\alpha = 1$ – along the edge of a foundation, $\alpha = 2$ – at the top of a structure (a change in α at intermediate levels is assumed to be a linear dependence); for anchor bolts, bracings of the supporting parts of temporary structures (with the exception of bracings of wooden structures) $\alpha = 5$; for the remaining structures and elements, $\alpha = 1$.

This method of determining the seismic loads can also be applied to the majority of mine structures, located in the areas of blasting on sites adjacent to the seismically dangerous zone, outlined by r_c.

Multi-story buildings can be considered as another group of structures on the surface of mine works. Accordingly to the effective standards and codes of building in seismic regions, a rated seismic load S at site k of a building with a distributed load there, a mass

having a weight of $Q_\text{н}$ can be determined at a Ball reading of seven seismicity, as

$$S_{ik} = \frac{1}{30} Q_\text{н} \beta \eta_\text{к}.$$

where β – coefficient of dynamicity depending on the value of the period of free vibrations of the structure, T and equal to $\frac{0.9}{T}$; the amount of β is taken within the limits of 0.6–3; $\eta_\text{н}$ – a coefficient, depending on the character of deformations of the structure during its free vibrations and at the site of the load $Q_\text{н}$ in the mine construction area.

The values of $\eta_\text{н}$ are determined according to standards and codes of construction in seismic regions in accordance with the horizontal deviations in a structure during its free vibrations at the considered site k and at all points i of the distributed masses in the system.

During the calculation of stone buildings up to five stories having cross walls, spaced up to 12 m apart the product of the coefficients $\beta\eta$ for the first-fifth floors are assumed according to the standards equal to:

In a one-story building	3.8
In a two-story building	2,7-3.8; with an average 3.2
In a three-story building	1.9-3.3-3.8; with an average of 3.0
In a four-story building	1.3-2.4-3.2-3.4; with an average of 2.6
In a five-story building	1.0-1.8-2.5-2.9-3.0; with an average of 2.2

During the calcuation on a local seismic load of the walls, including the wall fillings of the superstructure and their bracings to the superstructure, the value $\beta\eta$ is taken for the corresponding level of the superstructure, but it is not less than 2.

In the calculation of parapets, towers of small cross section on buildings, the frames of balconies, small bracket plates, anchor bolts (except for the bracings of wooden structures) the value $\beta\eta$ is assumed to be equal to 5 at an inefficient horizontal or vertical direction of the seismic forces.

However, the indications of standards proposed here by definition of the seismic loads of buildings during earthquakes should not be completely extrapolated to the calculation of the mine buildings, located in the regions of blasting because this will result in a substantial overrating of the loads.

In operations, proposed as the basis of effective standards and codes of construction, buildings and structures are considered, which are located in the region of an earthquake and which sustain considerable damage. A comparison of calculations, made according to an earlier enacted proposal based on construction in seismic regions, with calculations made in accordance with effective standards, shows, that in the second case, allowing for the above shown coefficients of $\beta\eta$, the calculations and the actual state of the damaged buildings will agree more accurately, but the seismic loads of the buildings exceed the loads, obtained on the earlier established position by approximately two times.

It was proven that the loads, found in accordance with the earlier enacted proposals for regions of earthquakes, were underrated. Nevertheless in buildings and construction, located in the areas of blasting, but beyond the limits of the seismically dangerous zones calculated and engineered in accordance with that proposed under an arbitrary Ball scale seismiticy of number 7, no substantial damage was observed.

Thus, in rated buildings, located in the areas of blasting beyond the limits of seismically dangerous zones (the zones, described by a radius of r_c), the seismic loads, determined at a Ball scale seismicity

of number 7 accordingly to the effective standards, one should be reduced by means of introducing a coefficient less than unity; the tentative value of this coefficient as a first approximation is equal to 0.5. In this instance, for example, the average values of the product 0.5 $\beta\eta$ for stone buildings having a height of 1-5 stories amounts to 1.9-1.1, and on the average – for a three-story building – 1.5. In this case, the value 0.5 $\beta\eta$ for the first story will be ≈ 1, and for the third (upper) story $1.9 \approx 2$.

The determined seismic loads of the buildings in this instance are quite close to the seismic loads, found according to the method of calculating loads for man-made structures, at the same seismicity and with a coefficient $\alpha = 1$ along the edge of the foundation, and $\alpha = 2$ at the top of the structure.

Considering what is proposed prior to conducting the special investigations, it is recommended that one should determine the seismic loads of buildings in the areas of blasting arbitrarily in the same way as the load of a man-made structure, considering the seismicity (predominantly at a Ball scale of 7), characterized by the locality of the building relative to the blasting site and by assuming that α = 1-2.

The seismic forces in conjunction with other loads pertain to specific pressures.

Buildings and construction can be computed based on the simultaneous action of working loads, dead weight of the structure and seismic forces. A wind load, that discharges the effect of the friction forces, the dynamic action of the equipment, the braking forces and horizontal impacts of the mobile composition, the braking effort of the cranes, the inertial forces from loads, raised by ropes on flexible suspensions, are not considered based on seismic forces during the calculation of the structures.

In the calculation of man-made road construction for seismic activity the braking forces and the horizontal impacts of rolling stock, the wind load, the effect of temperature, the pressure of the ice and vessels are not considered.

The direction of the seismic forces can be taken as predominantly horizontal, acting lengthwise or transverse to the construction since when calculating and designing structural elements the least effective direction of the forces is taken. The weight and pressure of the soil during the seismic calculations are considered under the assumption of a diminished value of the angle of internal friction of the soil. The strength of the structure is checked in the presence of and in the absence of a temporary load.

During the determination of the quantities $Q_{\text{н}}$ (the loads, which cause inertial forces) the dead weight of the elements of the buildings and structures and of cranes is based on normal loads without overload factors, but working and snow loads – at normal loads with a coefficient of 0.8.

In the calculation of structures of the warehouse and silo types normal working loads are fully considered. In the determination of seismic pressures in the blasting zones for mine loading bins and a number of bin superstructures and tunnels, the values of normal seismic loads are taken in accordance with the data, presented in Chapter XI.

The calculation for the strength of steel and wooden structures apart from the coefficients of operating conditions taken in accordance with the construction standards, and in view of the short time effect of the seismic load, also takes into consideration an additional coefficient of operating conditions, equal to 1.4. For stone, concrete and ferroconcrete structures this coefficient is taken to be equal to 1.2, but for the prestressed reinforced concrete it is 1.

Construction located in the areas of large scale blasting with regard to a construction relationship, should conform to standards and building codes in seismic regions at a Ball scale seismicity reading of about 7. Thus, in circular zones, delimited by the inner radius, equal to the radius of the seismically dangerous zone r_c, and with an outer radius of about 3 r_c, the utilization of uniform structures in construction as a dynamic ratio is recommended. Arched designs can only apply in the presence of a reliable base. Beam spanning structures should be attached to supports, sectional structures of a construction – unitized. The weight of the structure, especially the weight in its upper part, and the weight of the spanning structures should be of maximum shortness.

The enumerated requirements are, to the least degree, applied to metallic and wooden structures, and also to combinations, for example, metallo-ferroconcrete structures at the site of reinforced concrete in the lower zone. Of the remaining structures in all cases ferroconcrete are preferable. Stone and concrete structures should not be used.

Guidelines as a part of the planning and selection of structures of earthquake-proof buildings and construction are as follows.

Longitudinal and transverse walls should be symmetrical relative to the axes of the building, the configuration of the building in the plan should be very simple, and one should avoid breaks in the walls in the plane. Inner transverse and longitudinal walls should run the entire width or length of the building, the partitions should possibly be wider, of the same for and evenly spaced; in two-story and higher buildings. one should avoid internal concrete and stone posts.

Expansion and contraction joints should be antiseismic. The width of the joint – not less than 30 mm; at a height of more than 5 m, the width increases to 20 mm for every 5 m of height. In

buildings with support walls, the joints are created by erecting paired walls, with support columns or abutments – by erecting paired frames, columns or abutments. Combinations of a wall or abutment are possible. The erection of paired columns on the overall foundation is allowable.

Foundations of a building or a seciton of it between the joints should, as a rule, lay at one level. Under the support stone walls one should use a predominantly continuous footing. With sections of continuous footings and walls of a basement made of heavy units, a masonry bond should be provided at each corner and intersection. With a device under the walls of individual columnar foundations one should mutually connect them, so that the foundation beams can be spanned by continuous ferroconcrete beams and a good connection of sectional foundation beams should be provided for.

With slightly compressible dense clay, loamy, sandy and sandy loam soils, for buildings with a rated seismicity Ball scale reading of 7 and 8 at the top of sectional foundation plate along the entire perimeter of the walls in the layer of the brand span 50, one should make the steel framework as four longitudinal rods 8-10 mm in diameter, connected at intervals of 400 mm by transverse rods 6 mm in diameter.

Moisture proof interlayers in stone walls should be filled with a cement solution.

The height of stone buildings using masonry of the second category (slag blocks, brand 50 bricks and even brand 25 for the span) should not be more than 18 m (16 m)[1] at a seismicity Ball scale reading of 7 (8).

The height of the floors using masonry of the second category depends upon the seismicity and should not exceed a Ball scale reading

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[1]The first figure for number 7, and brackets for the Ball scale reading of 8; the same designations apply further on.

of 7 (6) at 7 (6) m; the ratio of the height of a floor to the thickness of the wall in this case should not exceed 14 (12). The critical distances between the axes of walls, between frames or between abutments are limited to 20 (16) m.

The joining of walls in basic mine buildings and related construction located in on areas, adjacent to a seismically dangerous zone, should be strengthened by short paired straight or curved rods having a diameter of 5-6 mm, extended 1.5 m from the joinings. It is recommended to install the paired rods at 1 m in height and to connect them with transverse rods 4-5 mm in diameter at each 400-500 mm interval. Instead of the outlined design for the walls over their entire length in the joints over the window cross members and lower window sills, one can use two continuous rods, 5-6 mm in diameter connected to one another at 400-500 mm intervals by transverse rods 4-5 mm in diameter. This design is especially expedient for small buildings and structures, located beyond the limits of 2 r_c of antisemismic belts in lieu of those buildings and structures described below.

The masonry work filling the building frame should have a thickness of not less than 120 mm in joinings made of steel reinforcement as having a diameter of 4-6 mm and a length of not less than 0.7 m, fixed at 700 mm by height, and attached to the uprights of the frame. These joinings should be sealed with masonry over their entire length. Furthermore, at every length of masonry of 1.5-2.0 m, an upper span piece should be erected. The specification number of the masonry filler span — should not be below 25. Similar joinings should also apply to brick partitions. Masonry fillings of framework and partition based on ratio of the length to the thickness equal to 20 and more, should be reinforced.

Self-supporting stone walls should be erected using mortar ranked not lower than the second category and should be joined to the frame work by tee-anchors made of a band of steel in the joints of masonry work.

The number of anchors should be not less than two for each 10 m^2 surface of wall, and the area section of each anchor – not less than 1 cm^2. Above and below the anchors in the masonry joints there are reinforcement rods, 5-8 mm in diameter and 1.5 m in length with a cross-sectional area at every joint of not less than 0.5 cm^2.

In buildings with walls made of *heavy units* the masonry bonding of the units in the corners and at the intersections of the walls should be provided for. In retaining walls the joining of the units with ferroconcrete keys, which are included in the recesses of the units, or in the joining of the units by means of the welding the matching parts is permissible. Furthermore, in the joining of the walls reinforcements should be included in the horizontal joints. The horizontal joints between the units are filled with a cement plaster mix, the vertical ones – with concrete of a specification number of 50 and above. It is recommended that *the panels of the walls, partitions, over-laps* of large-panel buildings be mutually connected with the aid of matching parts, protected from corrosion by concrete or by a cement plaster mix.

The width of the piers in the masonry work of the second category should be not less than 750 (1200) mm, and the width of the apertures – not more than 3.5 (3.0) m.

Parapets with masonry work using a cement plaster mix are permissible up to a height of not more than 1.2 m, taken from the top of the over-lap. The stability of the parapets and cornices should be assured with the bonding.

Cross numbers are arranged along the entire width of the wall (or less than 12 cm apart); as a rule, they should be ferroconcrete or reinforced stone using a cement plaster mix with a specification number of not less than 50 with a steel framework made up of four rods, 5 mm in diameter, arranged in two joints of a straight arch and embedded in piers not less than 0.4 m deep.

Antiseismic straps. In the area of blasting in the main mine buildings and construction activity, located at sites adjacent to a seismically dangerous zone, it is recommended that antiseismic straps, and especially reinforced stone bands of a facilitated design be used. Beyond the limits of the area described as having a two-fold radius of a seismically dangerous zone ($2r_c$), antiseismic straps in small construction operations and buildings of an auxiliary character cannot be arranged feasibly. In the latter case the reinforcement of the walls usually amounts to merely placing the cement over and under the apertures of continuous paired rods.

The monolithic ferroconcrete embedded in the walls over-laps or the bracket beams of the framework, and also the unitized (see below) sectional ferroconcrete over-laps and coatings replace the antiseismic straps. Monolithic over-laps should be tied in with the masonry work of walls using a metallic mesh with transverse steel reinforcement spaced 200-250 mm apart, placed in the wall along its whole length with depth and in the slab not less than 0.4 m deep.

The antiseismic straps in the absence of over-laps should be specified based on the height at a distance not exceeding the limiting height of the floor.

Antiseismic straps should be predominantly laid out at the levels of the ties. With walls 0.5 m wide and more, the width of the antiseismic strap can be equal to the thickness of the wall minus 12 cm, and with less thickness of the walls, the width of the strap can be taken equal to the thickness of the latter.

In the zones of the effect of blasting, in two joints of a steel reinforced brick strap, it is sufficient to insert longitudinal steel reinforcements consisting of four rods, 6-8 mm in diameter in every joint, tied in at intervals of 400 mm by transverse rods, 4-5 mm in diameter.

In coarse block buildings, reinforced block ties, aligned along the perimeter of the entire walls and tied in by means of the welding of matching parts at two levels or by means of unitized reinforced loops, are used as straps. The coupling of the antiseismic straps with masonry made of heavy units is attained by using an embedding device in the straps of the vertical steel framework having a diameter of 5-8 mm in the joints between the units.

When using sectional ferroconcrete antiseismic straps their couplings are welded along two planes or are joined by means of unitized reinforced loops.

Coverings and cappings of earthquake-proof buildings are made rigidly according to plan and are sealed along the contour. The floor beams are anchored in the walls or in straps by as much as through 2 (1.5) m; all of the girders (main beams) should be anchored. One should not space the beams closer than 0.8 m, and should align them along longitudinal grooves. The latter are permitted only for plates and should be thoroughly sealed with concrete or a mortar. The support for the ferroconcrete flooring on the bearing walls should not be less than 12 cm.

As sectional span designs one should use predominantly heavy panels with their support on the bearing walls or unitized spans as a substitute for them. The latter is done by means of ferroconcrete belts and framings by anchoring the beams in the strap along with the filling of the seams between the panels with a cement mortar, and in the absence of antiseismic straps — by introducing binder between the panels.

In the presence of straps based on standards for support walls monolithic ferroconcrete frames having a width of 120 mm with four rods 12 mm in diameter are used. The frames are hooked up with the ferroconcrete straps, resting on the support walls. By connecting the beams with the frames vertical superstructures result, resting on the seams between the panels of the beams.

In the zone of the effect of blasting, a unitized capping according to the first method is assured by a predominantly ferrobrick strap using a number 50 mortar with longitudinal steel framework in not less than two joints. Four rods having a diameter of 6-8 mm are placed in every joint seam, and are connected at an interval of 400 mm by transverse rods 4 mm in diameter. The panels for the flooring should be connected by clamps to one another and also with straps and should be unitized. In order to connect the flooring with the straps, vertical frames or single rods are inserted in the joints between the panels. In all cases the laying of the flooring is done using a grade 50 mortar, and all joints, grooves, voids are thoroughly filled with concrete or grade 100 mortar.

In the zone of the effect of blasting the tying of the flooring according to the second method can be done at one level by means of welding the matching parts using cover plates. The matching parts are installed on the surface of the flooring along the lateral edges at a distance of 3-6 m.

The welding of the large flooring panels to the matching parts of transverse or longitudinal beams by filling the joints between the panels with mortar cement is rather an effective antiseismic measure. With the support of the large-size flooring plates on the walls the welding of the plates to matching parts, in certain cases, is used for their support to metallic straps anchored in the masonry.

By assigning a sum to the described measures, specified for the design and construction of the buildings and related work, located near the seismically dangerous zone, described as r_c, one should note the principal ones.

1. The structural design requirements, in particular the location of capital walls, the selection of height of buildings and related construction, the height of floors; the over-all sizes of walls, of piers, of openings can be worked out.

2. Short paired rods can be inserted in the unions of walls.

3. Reinforced stone straps of a facilitated design (excluded in special cases) can be used.

4. Plates and floor beams can be unitized, capped and anchored.

In small mine construction operations and buildings of an auxiliary character, located predominantly beyond the limits of the areas described as the doubled radius of a seismically dangerous zone, and within the limits of 3-4 r_c, the antiseismic measures (under rigid structural design requirements) amount to the following.

1. Along the contour of the walls above and below the openings, continuous paired rods made of rolled wire are inserted.

2. The plates and floor beams are unitized, capped and anchored.

During the operation of fulfilling the antiseismic measures, specific attention should be given to the need to provide, in all cases, for reliable cohesion of concrete with mortar, the reliable cohesion of concrete with masonry and steel reinforcement. The need for the latter has been confirmed by the inspections of damaged and destroyed buildings and related construction in the regions of earthquakes and in the zones of blasting.

The above presented requirements for construction are based predominantly on tests of the performance of buildings with relatively heavy walls of stone masonry: concrete and stone retaining walls, tunnels, culverts, bridges and other structures of massive design. At a large weight the construction is correspondingly great, along with the seismic forces, and the construction measures designed for the protection of the construction against the destructive action of blasting also increases in cost.

The designs of the buildings and related construction should be characterized by small seismic loads, should have a minimum intrinsic weight. This requirement should be met not only in especially unfavorable cases of seismic loads but should be also required for the majority of mine construction located in the regions of blasting.

The experimental design of four- and five-story structural frame buildings with walls made of warm ply ferroconcrete panels showed that the utilization of the light-weight panels lowers the seismic loads (in regions of earthquakes) by 2-5 times in comparison with loads on the support stone walls.

With a reduction in the weight of the construction, and primarily the weight of the walls and flooring, the values of seismic loads approach the values of wind loads and the above described antiseismic measures to a large degree become unnecessary, which guarantees a corresponding reduction in the cost of the construction.

3. The Protection of Constructions on Underminable Sections

As a result of excavating mineral deposits the earth's surface is altered. When dealing with large scale, especially steeply dipping, mineral deposits troughs, funnels, cracks and other features (Fig. 36a, b) form on the surface through systems of cave-ins.

When working in gently dipping layers of mineral deposits, which are of local occurrence and which also are located in rather deeply bedded layers, shifting basins are formed (Fig. 36c, d).

The boundaries of zones and shifting basins are set by the rules of protecting the construction. For the Krivoy Rog deposits, with a coefficient of consolidation of bedrock in a hanging wall exposure at a scale of M. M. Protod'yakonov's within the limits of

1 (clays) to 4 (sandstone, weakly consolidated) the values of angle β (Fig. 36a, b) are assumed to be equal to 45°; angle δ — 75°, angle γ commonly corresponds to the angle of incidence of the underlying rocks of the flat side.

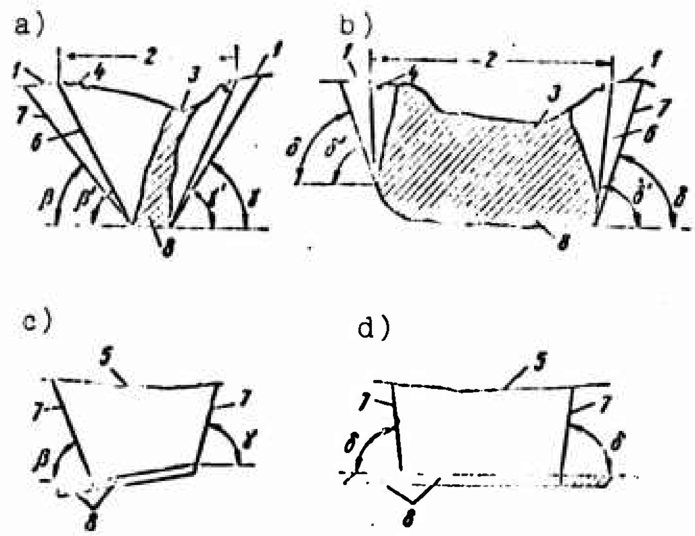

Fig. 36. The zone of caving and shifting of the surface: a — a cut transverse to the stretch lay of the ore deposit; b — the same cut but along the lay of the deposit; c — a cut transverse to the lay of the stratified layer; d — the same cut but along the lay of the layer; *1* — the zone of shifting; *2* — the zone of caving; *3* — dips, funnels; *4* — fissures; *5* — shifting basins; *6* — the surface of caving; *7* — the surface of the shifting; *8* — mineral deposit.

For ratner well consolidated rocks of a hanging wall ($f \geq$ 5-6) — sandy schist, sandstones of iron ores — an angle β' is assumed to be equal to 45-55°, and at the limited length of the deposit along a stretch of the upper horizons (70-350 m) 60-70°, angle δ' = 85°, and an angle γ' also corresponds to the angle of incidence of the

underlying rocks of the flat wall. The angles of caving and shifting in the alluvium and in limestones are taken as 50°. By using the shown angles and by following other mine-surveyor data, one determines the boundaries of the zones of shifting and caving and one designs the construction of preventive (guarding) pillars, within whose confines the mining of the deposit cannot be done. The guarding pillars are so located that the buildings and the related construction being protected cannot occur in the zone of caving. As a result of the insufficient overall control of the physical-mechanical properties of the rocks affecting the values of the angles of shift, one makes a certain allowance, in the construction of pillars, and the pillars are not erected along the mid-town of an industrial park, but at a certain distance from it, using a width of preventive berm, equal to a distance from the building or related construction to the border of the zone of shifting. The width of the preventive berm for the mine shafts, mine pile drivers, mine surface buildings and buildings of lift machines should not be less than 20 m, but for small auxiliary buildings – 10 m.

The data, necessary for the construction of pillars, are taken based on the rules for the protection of buildings and related construction, worked out for all mining and coal basins using the calculation of local geological mining conditions. The following angles of shifting were established (Fig. 36c, d) by the numerous observations of the shifting of the surface, produced in developing the comparatively deep layers of small thickness in the coal basins. The angles β, γ, δ (in the absence of irrigation and quick ground) in the deposits are equal, on the average, to 50° (they change from 45 to 60°). Depending on the physical-mechanical features of rocks the angles of shift in bedrock constitute 55-85° with a dip of the beds of 0-5°; with an angle of dip of the beds of 30°, the angles δ fluctuate within the same limits, angles β – from 50 to 70°, and angles γ have values from 70 to 90°.

In the absence of backing the made space, the amount of settlement at the surface constitutes 40-60% of the thickness of the layer.

One ought to note that a great number of various factors have an effect on the magnitude of [illegible] the main ones being: the thickness of the mineral deposit, its bedding depth, the angle of dip, properties of the rocks, the applied systems of exploitation, and so on.

The undermining of buildings and related construction usually leads to their complete failure, or in any case, to considerable deformations. For example, in undermined one-story stone building having a length of 35 m with a settlement of the surface of 0.2-1.4 m, sustains very serious damages: in longitudinal walls a series of cracks appear with the greatest opening of the seams in an upper part, up to 600 mm. In another case, an industrial structure having a length of 80 m, a width of [illegible] m and a height of 15 m with brick walls and continuous footings, with a coating of sectional plates along the metallic girders divided lengthwide into two parts by a settlement seam, was undermined by two converging layers with a ratio of the working depth to the sum total thickness of the layer equal to 180. As a result of the undermining ground cracks and a series of fissures of a width of [illegible]-30 mm in the walls of the section nearest to the mining work. A settlement seam was opened by 150 mm, the walls inclined by 100 mm, a parting line developed in the settlement seam in longitudinal and transverse directions by 80-120 mm; the difference in the mark of the parting line was 130 mm, the void space was wedged, the deformations of the girders were noted, the practical danger of the dip of the sectional plates of the flooring appeared.

The expected deformations of an earth surface can be determined in a calculated way. Accordingly to Provisional Technical Specifications (VTU-01-58) the maximum relative horizontal deformations of the earth surface with gently sloping and inclined bedding planes are equal to 70% of the ratio of the thickness of the layer to the working depth (0.7 m/H), but the maximum slope in the shifting basin is equal to 1.5 m/H and so on.

Accordingly, using the same technical conditions four categories of the ground, characterized by the expected deformations of earth's surface (Table 3) have been established.

Table 3.

Category based on the deformations of the earth's surface	I	II	III	IV
Limits of the changes of relative horizontal deformations	$(8-12)\times 10^{-3}$	$(5-8)\times 10^{-3}$	$(3-5)\times 10^{-3}$	$(1-3)\times 10^{-3}$
Limits of the changes in slopes	$(10-20)\times 10^{-3}$	$(7-10)\times 10^{-3}$	$(5-7)\times 10^{-3}$	$(3-5)\times 10^{-3}$
Limits of the changes in the radii of curvature, km	1—3	3—7	7—12	12—20

With small deformations (with a relative horizontal deformation of less than $1\cdot 10^{-3}$, etc.), special measures of protection are not necessary. With the large deformations, extended beyond the limits shown in Table 3, the protection of the construction is technically inadvisable. Construction should generally not be located on such grounds.

The most important *construction measures* for the protection of construction activity, established in accordance with existing present day experience of undermining of residential and, in part, industrial buildings, can amount to the following.

Buildings and construction should be divided lengthwise into separate units using *deformation joints* having a width of 60 mm. The distance between the joints is established also based on the calculations for one-story buildings with brick walls, in a number of cases, assigned within the limits of 15-25 m, and for structural frame buildings and related construction up to 40 m; the distance between the joints can be increased with an increase in the rigidity of the walls in their plane, with an increase in the bearing capacity of the soils and with a decrease in the loads and in the weight of the overall structure.

Within the limits of the joints the basic support designs of the units should be made in the form of spatially rigid boxes, rigid frames with diaphragms, and so on.

The *walls* should be arranged throughout the entire width and length of the unit. Units should be joined in a plane with transverse walls and frames, which are spaced not further than 3 m from the end of the unit.

The location of the walls in the plane is taken based on the possibility of symmetric axes relative to the building or related construction, and one ought not to allow fissures in the walls in the design. In buildings with support stone walls on continuous footings it is necessary to avoid columns with separate foundations inside the building.

The weight of the buildings and related construction, independent of the specified construction measures, should be minimum, which can be achieved by utilizing contemporary large panel wall fillings with effective heat insulation, and also with the reduction in the weight of the flooring and cappings.

The distribution of the weight of the structures within the limits of the length of a unit and pressure on the soil in the plane of the foundation should be uniform. The height of the walls also based on the feasibility of foundations within the limits of a unit, should specify identical joints, but with a difference in the heights of the sections of more than 2 m, it is necessary to employ a settling joint device. Basements should be so arranged to have spaces between the joints and to include one or several whole units.

The projections of the masonry of straps and of cornices, as recommended, should not be spaced more than 250 mm, and within the limits up to 0.4 of the thickness of the walls.

The width of the partitions should be not less than 1.2 m; the breaks in the masonry whose sizes are limited, are recommended to be spaced not closer than 1.5 m from the edge of the unit and 0.6 m from the transverse wall.

The window lintels in the stone walls should be predominantly of ferroconcrete along the width of the walls with a restraint of not less than 250 mm. The height of the parapets should be taken as not more than double the thickness of the masonry.

The brick masonry is bounded by a course of headers through 4, and slag slab masonry – through 2 of a course of stretchers, with bonding in each course at angles and joined to the walls.

The foundations of the construction as recommended, should be made of ferroconcrete. Continuous footings and foundations of rubble masonry relative to horizontal deformations of less than $4 \cdot 10^{-3}$ and $2 \cdot 10^{-3}$, are permitted. Abrupt weakenings of sections of the foundations are not allowed. Openings with a side greater than 0.3 m are reinforced. The width of foundations is recommended to be taken along the width of the walls using the necessary plates for the support of the base and for the formation of the crown.

The transverse sections of continuous footings are allowed to be right angled (with a widened supporting plate and without it), by trapeziform, tee-shaped (with a shelf above) and wedge-shaped. However, the data about the utility of wedge-shaped and tee-shaped foundations are discrepant, and there are direct indications on the infeasibility of their utilization. Under practical conditions various pressures on the plane of the wedge, and because of this, the appearance of considerable amount of additional horizontal loads on the foundations according to value are possible, which results in an increase in the expense of the steel framework, being erected lengthwise to the vertical face of the foundation. In any case a failure of a right angled form with supporting plates or without them on conventional foundations should be especially established.

The utilization of sectional foundations under a condition of bonding 1/3 of a unit for each course with the bonding at all angles and joining to the walls. The junctions of the sectional foundations should be unitized.

Waterproofing along the edges of the foundations can be accomplished predominantly by using a dense mortar. Waterproofing using rolled material [sheets] is not allowed.

Strengthening the walls and foundations is mandatory, if they are made of brick or stone masonry, concrete or a rubble concrete. Strengthening is attained due to the utilization of special steel framework. In the joints of walls 1-1.5 m in height and 120-150 mm in the thickness of the masonry, the steel reinforcement having a diameter of 4-6 mm, is set in 1.2-1.5 m from the intersections of the walls. Apart from the local reinforcement, in accordance with the special technical conditions, in the necessary cases continuous steel framework is installed. In this case they consider that the walls are subject to vertical loads, which appear during uneven settlement and the foundations, furthermore, should be acted upon by horizontal loads, induced by horizontal deformations of the bases.

The strengthening of the walls is produced by introducing reinforced stone or ferroconcrete straps along the perimeter of longitudinal and transverse walls, usually located at the level of window lintels and window sills. The reinforcement of the reinforced stone straps 4-8 mm in diameter is spaced at a distance of 50-150 mm based on the thickness of the walls, in a layer of a mortar of a grade not less than 50. The distance from the reinforcement to the crown of the masonry of the window sills should be not less than 140 mm, but the height of the cornice strap of masonry — not less than 500 mm. Ferroconcrete straps are set at a height of 80-300 mm and a width, equal to the thickness of the walls or less than the thickness of walls by 120-200 mm.

The strengthening of concrete and stone foundations is done by means of a reinforced concrete frame strap having a height of 250-500 mm with the reinforcement near the base usually located lateral to the face of the strap.

The calculation provides for the technical conditions based on the increased pressures in the plane of the bases. In this instance with the exclusion of a part of the foundations from bearing as a result of undermining and uneven settlement of the base, the settlement of the building occurs over a relative short time. The increased loads on the bases become possible because of the strengthening of walls and foundations, and they are initiated depending on the degree of rigidity of the walls, which is determined by the ratio of the length and height of a unit, larger or less than 1.5. The calculated pressures are taken equal to the calculated resistances of bases with the introduction of a coefficient which takes into account the increase in the calculated pressure. The value of this coefficient is equal to 1.2-1.5 for gravelly, coarse and medium sands; for loams and fine sands, 1.5-1.8; for clay loams and clays, 1.4-1.75. The large values of the coefficients correspond to the high rigidity of walls and they can be assumed in the presence of the settling joints in the closing walls and incontinuous [solid] interior walls.

In structural frame construction and buildings it is necessary to resort to the connecting of columns in each compartment; in a number of cases are provided the connecting of the foundations, the possibility of straightening out the crane runway girders in horizontal and vertical directions with corresponding increase in the overhead crane sizes. The filling in the walls should be fastened to the columns of the frame structure with flexible connections.

A unified method of calculating the designs of buildings and construction being undermined still does not exist. One of the simplest proposals of the given problem is given below.

The undermined constructions are regarded as beams on two supports, or as cantilever beams. The latter is the most characteristic scheme of loading. If we examine this case, then the force action on the undermining of the construction is reduced to the following.

With the settling of the surface in section *1-2* of the length of construction (Fig. 37), the average pressure on the plane of the base, which constitutes to precipitation the amount σ_0 within the limits of this section up to settlement, is practically reduced to zero. Simultaneously, the value of the average pressures in the plane of the base in section *2-3* is substantially changed and it attains a maximum value of σ_1 at point *2*, which corresponds to the limit, beyond which there is a cutting of the soil under the sole of the foundation and a certain leaning of the construction. The latter corresponds to the decrease in the cantilever l_1 and the value of pressure at point *2*.

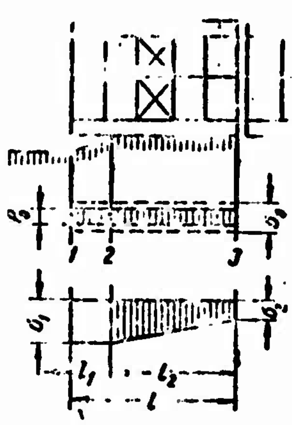

Fig. 37. Pressure on the base of undermined structure.

By assigning a value of maximum pressure σ_1, one finds the value of cantilever l_1 and the greatest value of the bending moment M_2, based on the value of which it is possible to judge the rigidity and strength of the construction and the permissible length of unit, l. If the value M_2 is excessively great, one ought to increase the

number of stress-strain joints, by having shortened the same length of unit l, cantilever l_1 and correspondingly the value of the bending moment. With the calculations one ought to consider the limiting values of the cantilever overhangs of the foundations l_1 during the undermining of the buildings, which according to the data in practice, reach 15-20% of the length of the unit.

Investigations according to the determinination of the value of the lateral horizontally directed pressure on the foundations during undermining, were conducted on top of the building, carried out in accordance with technical conditions, VTU-01-58, using two units (sections) each having the size, 12 × 15 × 8 m. One unit has two ferroconcrete straps in the walls and two straps in the continuous footings. The span of metal to the reinforcement of the walls and of foundations, 2.5 kg/m^3. The soils – clay loams of average density. The deformations of the earth's surface (base) as a result of mining activity were characterized by a radius of curvature up to 1.5 km, inclined up to 10 mm/m and with horizontal deformations up to 4 mm/m During the mining activity the greatest lateral pressure attained is 0.96-1.3 kg/cm^2 under a building. Following the mining activity the lateral pressure in the direction of the projection of the bed disappears, and crosswise to the projection of the bed a residual lateral pressure comprises about 0.5 kg/cm^2.

Using temporary technical conditions VTU-01-58 for calculating the undermined buildings with walls made of brick masonry and heavy blocks, a rather simple method of ultimate deformations of a structure is provided. According to this method the over-all sizes of the compartments and the degree of reinforcement are found separately for the walls and foundations on graphs, given under the technical conditions, depending on the curvature of the base and of the relative horizontal deformations of the surface during the undermining. Depending on the conditions, the spanning of the horizontal structural framework for strengthening the walls constitutes 0.15-2.0 kg/m^3 of the building, the spanning of the metal work into a vertical

reinforcement (used sometimes) is equal to 0.25-0.5 kg/m^3, and the spanning of the structural framework of foundations is found to be within the limits of 0.25-1.0 kg/m^3 of the building. The determined quantity of structural framework based on this datum is distributed (also separately) along the straps of the walls and foundations.

During the examination of a structure made of brick, having a length of 60 m, a height of 11.6 m and a volume of 7800 m^3, with a calculated resistance of the soil of 1.8 kg/cm^2, and of horizontal deformation of the soil of $5 \cdot 10^{-3}$ and the radius of curvature of 5 km, the following was established. In the partitioning of the building into two compartments by 30 m in size in order to strengthen it, reinforcements 1 m^3, are required for the building: in the walls 2.2 kgf, in the foundations, 0.8 kgf, the overall span of the structural framework on the whole building is 23.4 t. In the partitioning of the building into four compartments, 15 m in size, reinforcements 1 m^3 are required: for strengthening the walls, 1.5 kgf, foundations, 0.3 kgf, and the overall span of the reinforcement for the entire building is 14.04 t.

At the same conditions and with the height of the building at 8.3 m, the partitioning of the buildings into 2 (or 4) compartments of a length of 30 (or 15) m leads to the need for introducing reinforcements 1 m^3, in the buildings: in the walls 0.85 (or 0.22) kgf, in the foundations, 0.8 (or 0.13) kgf, and on the overall span of the reinforcement to a building, 8.8 (or 1.9) t.

In calculating the structural frame systems of structures and buildings it is most expedient to consider directly the deformations of the systems, which appear as a result of the effect of the horizontal deformations of the surface during the undermining and separately, with the effect of uneven settlements of the base (the curvature of the surface during undermining).

The horizontal deformations of compression or of tension at the base of the construction correspond to the displacement of the supports

of the uprights of the frame by the value Δ_{Γ}, equal to the product $0.8\ \varepsilon l_{ц.т}$, where ε – the relative horizontal deformations of compression or of tension; $l_{ц.т}$ – distance from the center of gravity of a system to the uprights; for a frame with a span L (Fig. 38) $l_{ц.т} = 0.5\ L$; 0.8 – the coefficient which takes into account a decrease in the intensity ε.

The curvature of the surface during undermining leads to the rotation of freely standing uprights with horizontal movements at their tops (in the absence of a cross-bar on the frame) by the amount Δ_A and Δ_B (see Fig. 38).

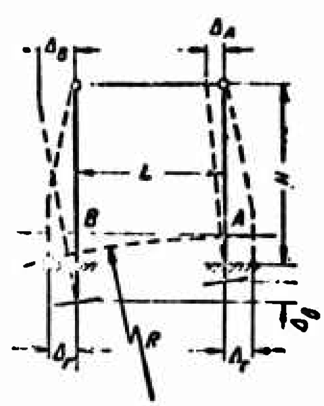

Fig. 38. Scheme for calculating the deformations of the structural frame of a building.

The difference in the displacements of the tops of uprights $\Delta_{\Pi} = \Delta_B - \Delta_A = L/R\ H$, where R – the radius of curvature of a surface during undermining.

The calculated displacements allow one to directly determine the strengthening, which appears in rods and joints of the structural frame of the construction of various systems during undermining.

4. The Protection of Construction in Areas of Mine Dumps

When exploiting mineral deposits by underground or open pit methods from shafts on open pits large volumes of waste rock

are excavated, which are stored in special sites on the surface – dumps. The tailings of production of ore processing combines, primarily crushing-grading and crushing-dressing plants (so-called "tails"), are also earmarked for the dumps.

The material and granulometric composition of the dumps can be highly variable. Dumps of tailings of crushing-dressing plants usually consist of crushed rock and partly of alluvial materials having a coarseness of 50-0 and 25-0 mm. Tailings frequently contain several kinds of ores and sometimes can be subsequently reprocessed for the purpose of a more complete extraction of the metal. Frequently, the crushed rock following grading, can be used for construction and other purposes.

The dumps of rocks issuing from open pit and underground workings consist predominantly of an almost barren mass.

For enterprises exploited over long periods of time (several decades), the dumps encompass considerable areas. Therefore, the question commonly arises about utilizing the dump as foundation material in construction.

As a foundation, the materials in dumps are heterogeneous, but test data and data on practical experience with the construction and operation of structures provide a basis for taking into consideration that the erection of a structure on a number of ten-year old mine dumps having a calculated foundation strength of about 1 kg/cm^2 is entirely possible. Some data on investigations of mine dumps as foundations are noted below.

The investigated fill soil on one of the industrial sites is predominantly clay and clay loam. The age of the dump – several decades; mechanical consolidation after this material was dumped was not done. Because of the relative instability of the surface relief, the clayey dump material is wetted and plastic, in places,

viscous; clayey sand fill material is characterized by less moisture content. In the dump material there are admixtures of gravel, cobble, and bouldery ore fragments, whose quantity reaches 40%, and more frequently, it constitutes about 10-15%. In mine dumps one rarely finds inclusions of relatively course stones up to 400 mm in size. The thickness of the fill constitutes 6-10 m. Given in Fig. 39 are graphs of the settlement of boring tests in experimental pits, obtained as a result of tests on soils by trial loads.

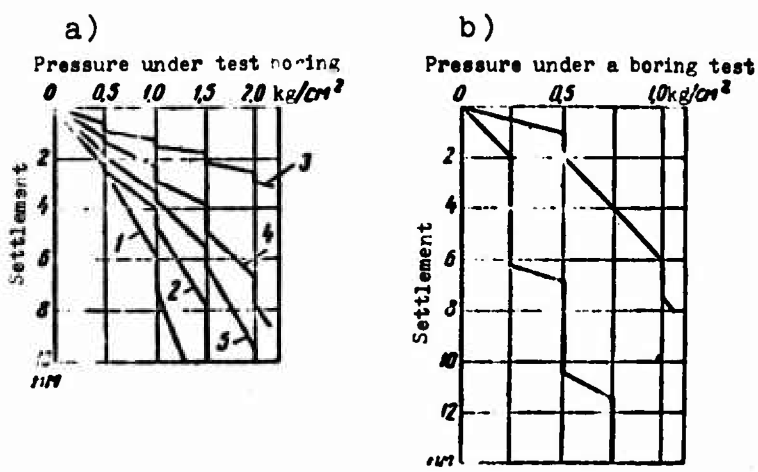

Fig. 39. Graph of the settlement of boring tests from the results of soil tests using trial loads: a) in clays and clay loams with admixtures of gravel, cobble and boulder-size ore fragments; b) in clay loams with admixtures of gravel sizes; *1* – clay and sandy loam, cobble-size fragments, up to 4%; *2* – clay, gravel and cobble-size fragments, 10-15%; *3* – coarse cobble-size fragments; *4* – silty sand, gravel and cobble-size ore fragments, 10%; *5* – clay loams, gravel and cobble-size ore fragments, 20%.

At another industrial site the age of the dumps at the time of the tests was about 10 years. The fill here was predominantly clay loam with some inclusions of cobble and gravel-size fragments. Graphs of the settlement of boring tests obtained as a result of tests on dumps by trial loads in experimental pits, are given in Fig. 39b.

In several industrial sites the investigations were made of mine dumps, 5-15 years old, consisting of syenite residium with the small inclusions of rock fragments. The thickness of the fill in one case constituted 5-10 m, in another, 2-6 m. In this case a certain part of the mine dump was unevenly consolidated by traffic moving adjacent to established railroad right-of-ways.

In Table 4, for example, data are given about the pressures under boring tests with equal settlement on the enumerated sites. The bearing capacity of the fill increases with an increase in the age of the mine dumps and with an increase in the content of rock fragment of cobble and gravel sizes and of ledge rock.

Table 4.

Age of the mine designs	Content of cobble and gravel-size fragments	Pressure during the settlement of boring tests, equal to 8 mm, kg/cm^2
5 years	Some inclusions	0.5-0.8
10 years	The same	0.5-1.0
15 years	"	1.0
Several decades	4%	1.0
	10-15%	1.5
	20%	1.7-2.0
	∿100% (coarse cobble-size fragments)	3.3

The characteristics are known of other fills, 16 years old (loess-like clay loams with inclusions of slags), used as foundations with a calculated bearing strength of 1.6 kg/cm^2 after consolidation by heavy tamping.

In general as a result of the differences in age, content of inclusions and degree of consolidation it is not possible to give unified indexes of the bearing capacity of mine dumps. The characteristics of mine dumps as foundations should be specifically established

for the studied mine site. There is a case for erecting individual mine operations on mine dumps. In particular, at the old Ural iron ore mine a surface building and haulage-ways of considerable length were constructed at the mine, a part of whose foundations were located on fill. These foundations are made of ferroconcrete slabs with each having a size by design, 14 × 4; 11.5 × 8 m, and others. The slabs were set on dump heaps 5- and 8-years old consisting of residual clay loams from bedrock with inclusions of rock fragments. The thickness of the dump heaps is up to 14 m. Beginning with the time of placement and loading of the slabs in 1952 and up to the present time the foundations continuously sustained loads at the average pressures in the plane of the foundations of about 1 kg/cm^2, and any unforeseen settlement of the foundations during the noted period was not noticed.

Recently, on this same industrial site mass utilization of the foundations, set on the mine dumps took place. This prevented a failure in the structure of the caissons and considerably reduced the cost of the building.

Therefore, utilizing the mine dumps as foundations instead of constructing expensive artificial foundations in a number of cases results in a reduction of the cost of the building.

However, when using mine dumps as foundations for the construction of buildings, preventive measures should be carried out. In the first place it is necessary to consider the possibility of uneven settlement.

This circumstance, which has a slight effect on the success of using wooden and metallic designs, acquires an exceptionally large value when selecting designs of ferroconcrete and even more so, for brick and stone buildings.

Ferroconcrete structural frames must be constructed so that the mutual connection between the uprights for separately supported foundations would be carried out predominantly with the aid of metallic

elements. The rigidity of the joints of the uprights with the girders in this instance are disregarded. The foundations using the limited sizes of the structures or their compartments in the design should be made in the form of rigid slabs and strips, at sufficiently rigid frames with heavy girders, erected at several levels; it is also possible to utilize ferroconcrete shoes.

Filling in the walls should be as light as possible. Stone designs in filling the frame, and also in the support walls, as a rule, should be excluded.

Elongated structures must be divided lengthwise into compartments according to settlement seams.

Generally, during the construction of buildings and related construction, located on settled soils, construction measures should be provided, which approximately correspond to those measures for the protection of undermined buildings and related construction.

Earlier mine dumps were considered from the point of view of their suitability as foundations for structures and buildings. However, it is necessary to keep in mind of that the nonore disturbed rock available either through mining, or as dump material of ore-dressing plants, or by means of working existing mine dumps, are frequently high-grade raw materials for the construction industry and, as a rule, can be directly used at the production site for the erection of mine structures. Specifically, construction road metal obtained as a result of crushing and sorting of disturbed bedrock of the majority of ore deposits, is characterized by high mechanical indexes, and is suitable for the manufacture of concretes of high grade, which correspond by strength to concrete with normal crushed granite stone.

The mechanical strength, for example, primary Urals hornblendites, Kachanar pyroxenites, Krivoy rog quartzites, Tagil' and other syenites, porphyrites and others, is very high.

Rock, which is almost inert due to a lack of disturbance over a long period of time, can also be successfully used for mine construction. Thus, for instance, with the utilization of concrete, made of crushed hornblendite rock, a hopper was erected joined to a support wall. From 1937 to the present time this construction was subject to severe conditions of operation, systematically undergoing the action of considerable impact loads, but no sort of deformation was received, which, in particular, can also be explained by the high strength of concrete, made from crushed hornblendite rock.

5. Special Cases for Protecting of Mine Construction at the Surface

Measures for protecting mine construction at the surface depend upon a number of conditions, and to a considerable degree, bear individual characteristics. Therefore, shown below are the characteristics for mine construction phenomena at the surface, which act unfavorably on their state of operation, and on the measures of protection.

During the excavation and shoring up of the shaft a considerable amount of excavated material is more or less involved; as a consequence of this, the shoring of the mouth of the shaft under the action of loads is displaced downward and corresponding settlement occurs as well as deformations of the machine stand and other elements of the pile driver, sometimes even adjacent to the construction. The same result can be observed during the settlement of shoring and the pile driver as a result of the settlement of the soil and other reasons. In these cases for pile drivers with guide pulleys the following measures of protection are taken:

1) Making the exception for the possibility of excavating rock by means of utilization the corresponding technology and systematic work when driving the shaft. All the excavation taking place should be tightly packed with rubble using the procedure in certain cases of subsequent tamping of the shored up space;

2) the transfer of the loads from the surface pile driver to the particular foundations;

3) the carrying out of construction measures, which prevent the unfavorable consequences of the excavation of rocks.

The development of the foundation area of the supporting rim of the shored up shaft more than is necessary, according to the calculated value determined, not allowing for the possibility of excavating the rock, is related to a number of conventional construction measures for the protection of metallic pile drivers with guide pulleys.

In certain cases the calculated area of the foundation is determined by reducing the calculated area of the supporting rim by the amount of the arbitrary area of the interior rim of the foundation. The width of this exclusive rim, depending on the nature of the soils and the cross-sectional area of the shaft is taken as 0.4-0.5 up to 1 m.

Sometimes, the area of the foundation of the supporting rim of the shored up mouth of the shaft is done by increasing its diameter; the latter sometimes is accompanied by the simultaneous filling of the shored up mouth of the shaft in the form of a truncated cone.

By joining the shoring of the shaft opening with the adjacent foundations, the base of the unit has a predominantly right angle form, which corresponds to the overall sizes of the construction.

In the cases of highly probable or inevitable sinking of the props the measures of protecting on the surface pile driver amount to the erection of supporting units of construction of a specific design, which allows for the movement of the pile driver in horizontal and vertical directions.

For the protection of the structures along with the multiple-cable lift and taking into consideration the possibility of their settlement and leaning the following is provided.

1) the erection of designed and laid-out foundations, separated from the shoring of the shaft opening; the erection of continuous flat, ribbed, compartmented and hollow foundation slabs, extensively used within the comfines of the construction areas and beyond its limits also in areas removed from the snored up shaft opening;

2) the utilization of artificial foundations;

3) the utilization of special designs, whose presence results in the possibility of eliminating settlement and leaning; the utilization of recess and slabs for the use of hydraulic jacks;

4) the utilization of worked out anchor pits or other designs, the presence which allows for the possibility of small movements of multiple cable lift machines, engines and deflector pulleys also in terms of the vertical direction.

CHAPTER III

STRUCTURAL DESIGN SOLUTIONS AND THE ELEMENTS OF MINE CONSTRUCTION

1. Structural Design Solutions of Mine Construction and Buildings

Mine construction and buildings frequently have, by design, complex forms. The main design problem consists of simplifying the outlines of the construction and buildings with their best possible approach to that of a rectangle and to simple units of rectangles. The second task is the *unification* of the design and construction solutions. This problem with respect to the utilization of mine construction and mine buildings should be examined séparately. A brief preliminary examination of the problem of the *unification of the plaster-like mix of the buildings* requires one to keep in mind possibility and expediency of maximum utilization of unified modular sizes and of corresponding unified sectional designs of industrial buildings for erecting mine structures.

For structural frames of one-story unified industrial buildings, without considering some of the exceptions, ferroconcrete designs are predominantly used today. The uprights of the girders are usually made of sectional ferroconcrete, set in sectional or unified shoes.

Ferroconcrete and stressed reinforced sectional beams and trusses of the floorings (spans of the frames) are installed on the uprights of the frames and hinged to them.

The unification of the sizes of the buildings is assured by taking the following measures.

For one-story industrial buildings it is preferable to utilize parallel spans with identical dimensions with respect to width and height. One should not allow a drop in the heights between the spans in one direction, if the magnitude of the drop is less than 2 m. The overhead crane size on the runways of an equal height with sectional ferroconcrete columns should be installed by the heaviest cranes, by using the upper crane arm at one level.

The sizes of the spans of the building should be multiples of the unit modulus, equal to 3 m (6; 9; 12; 15; 18 m), and with the larger spans - multiples of 6 m (24; 30, 36 m). Spans equal to 18; 24; 30 m are predominantly taken. The distance from the marked off axis of the overhead crane railway with a bearing capacity up to 50 t, as a rule, is taken as equal to 750 mm. The spacing of the piers is found to be equal to 6 and 12 m.

The height of the compartments in the nonoverhead crane type buildings and for suspension overhead cranes is taken as a multiple of 1 m, predominantly equal to 4-7 m. With overhead cranes the height to the overhead crane railway is also a multiple of 1 m, but at the height of more than 8 m to the overhead railway is taken to be a multiple of 2 m, and predominantly equal to 8; 10 and 12 m.

Multiple-story buildings are taken having a width of not less than 24 or 18 m. The grid of the columns in multiple-story buildings 6 × 6 and 6 × 12 m, the latter - with a payload on the flooring up to 1 t/m^2. The height of the floors of multiple-story buildings is taken equal to 3.6; 4.2; 4.8 and 6 m.

The drop in the heights between adjacent spans with a combination of longitudinal expansion joint and with ferroconcrete columns is solved using too columns, and without the combination of an expansion joint - on one column.

The outer face of the columns on the overhead crane runway with a load carrying capacity up to 30 t, as a rule, is aligned along the marked off axis; with a spacing of the ferroconcrete columns to 12 m and more, and also with the metallic frame, the outer column is located at a distance of 250 or 500 mm from the marked off axis. The center of the overhead crane parts of the middle columns is located lengthwise to the marked off axis. The end columns are offset from the transverse center line.

The tying in of the support walls is done in the following manner: in walls having a thickness of 380-500 mm the center line passes at a distance of 130-200 mm from the inner surface of the wall. With the support of the flooring beams on pilasters, the center line passes at 120 mm from the inner surface of the wall and 250 mm from the inner surface of the pilasters. At a support of the beams on the walls having a thickness of more than 500 mm, the distance between the center line and the inner surface of the wall is taken equal to 250 mm. The inner surface of the wall with the pilasters of more than 130 mm is located on the center line. With the support of the flooring slabs on the side wall the inner surface of the latter is placed 120-150 mm from the center axis.

Data on the unification on the volume-design and the structural solutions of buildings and related construction of coal mines and dressing plants are of interest. These subjects are associated with modular sizes, established from the position of the unification of designs, and basically they correspond to the latter data on the unification of industrial buildings. Data on the unification of buildings and related construction of coal mines include 39 dimensional schemes with the layout of these various full-scale

industrial structures. Diagram 4 (Fig. 40) can be used as a mechanical workshop, warehouse, garage, calorific plant. Diagram 15 corresponds to the placement of lift machines with the diameters of drums of 5 and 6 m. Diagram 27 (Fig. 41) characterizes the over-all sizes of boiler house with 2-4 boilers. Diagram 35 corresponds to a section of a commercial complex with a skip-winding lift producing 0.6-1.2 million t of coal per year and transporting the rock with the aid of a suspension cableway.

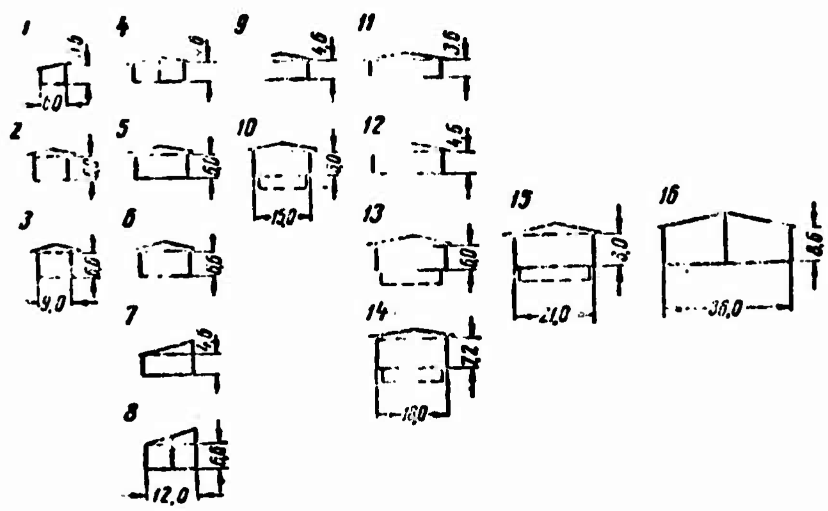

Fig. 40. Full size diagram of one-story buildings related construction.

Recently the first stage of work was conducted dealing with the ramiform unification of the design and construction solutions of buildings and related construction of the coal and mining enterprises. Unification up to now encompasses basically only one-story auxillary-production buildings of surface ore and coal mines, including ventilator buildings, warehouses, calorific buildings, maintanance workshops, electric substations, compressor and lift machine buildings. The overall sizes of the corresponding standardized sections of one-story buildings without overhead crane runways are given in Table 5, buildings with overhead crane runways - in Table 6.

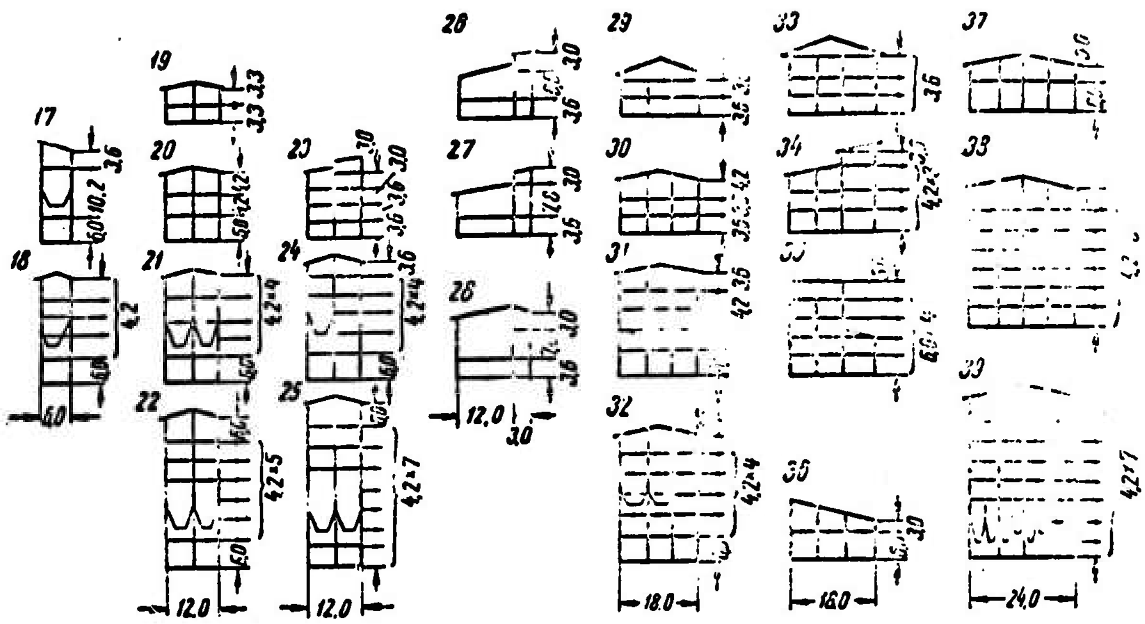

Fig. 41. Overall dimension schemes of multiple-story buildings and related construction.

Table 5.

Designation of the building	Number of spans	Sizes, m span	length	height
Ventilator building	1	6	9	5,8
"	2	6+6	9	5,8
"	2	6+6	12	5,8
"	1	18	18	5,8
"	1	18	30	5,8
Warehouse	1	12	6	4,0
"	1	12	12	4,0
"	1	12	18	4,0
"	1	18	12	4,0
Maintenance workshop	1	12	12	5,0
"	1	12	18	5,0
"	1	12	21	5,0
"	1	15	30	5,0
"	1	15	36	5,0
"	1	18	24	5,0
"	1	18	30	5,0
Clorific	1	12	6	5,0
"	1	12	12	5,0
"	1	12	15	5,0
"	1	15	6	5,0
Electric substation	1	15	15	5,0
"	1	18	15	4,0
Buildings for lift machines	1	15	12	5,4+3,6
The same	1	15	12	[illegible]
"	1	12	12	5,4+3,6
"	1	12	12	9,0

Table 6.

Designation of the building	Sizes, m				Load-lifting capacity of the cranes, t
	span	length	height to the overhead crane runways	height to the flooring	
Compressor	12	12	5	5,8	5
"	12	18	5	5,8	5
"	12	[illegible]	5	5,8	10
"	12	36	5	5,8	10
"	12	42	5	5,8	10
"	12	[illegible]	5	5,8	10
"	12	18	5	6,8	5
"	12	24	6	6,3	5
"	15	[illegible]	[illegible]	[illegible]	20
"	15	36	5	5,8	10
"	15	42	5	5,3	10
"	15	54	5	5,3	10
"	18	48	5	5,3	10
Buildings of lift machines . . .	18	15	5	6,6+3,6	20
The same	18	15	5	10,2	20
"	18	18	6	7,6+3,6	20
"	18	21	6	7,6+3,6	20
"	18	18	6	11,2	20
"	18	21	6	11,2	20

The spans of buildings are taken equal to 6; 12; 15; 18 m. The interval is 6 m. The height to the support flooring structure in building frames without overhead crane runways is taken in multiples of 1 m, in buildings without overhead crane runways but with support walls, multiples of 0.2 m. In buildings with overhead crane runways the height to the runways is a multiple of 1 m, and to the supporting structures of the flooring 0.2 m. The height of all basement compartments, 3.6 m. Lift equipment with a load-lifting capacity of 2 and 3 t is handled by single-beam suspension crane or monorails, but with a load-lifting capacity of 5; 10 and 20 t - ore overhead cranes. The buildings of lift machines are provided with basements and without basements. In the first case the height of a basement, equal to 3.6 m is set apart in Tables 5 and 6; in the second case the overall height of the building is shown.

The data given here are partly supplementary material during the transition from problems of prefabrication of buildings to problems of *prefabrication of mine construction,* which is presently found in the initial stage - the stage of development, the sum total of which still has not been established. Separate information on the problems of the prefabrication of various construction is given below, directly from their description. Relatively to the general recommendations in the trend of prefabrication of the design and structural solutions of mine construction to the nearest period can be shown as follows.

With this possibility one ought to use the data on the prefabrication of industrial buildings. For the construction, as a rule, one ought to use spans of 6; 9; 12; 18; 24; 30 m; mutual distances between the uprights, frames and supports, 6 and 12 m.

For the spacing of the uprights of piers, galleries and passages, one ought to take multiples of 6 m. At a small height or with considerable loads the spacing of the uprights may be equal to 6 m, for the remaining cases the spacing of the uprights one ought to assume predominately that of 12; 18; 24 m.

The spacing of the uprights of surface mine construction (deckhead buildings) is taken equal to 6 and 12 m, but in a number of cases it reaches 18; 24; 30 m.

The interval of the uprights of loading bunkers is usually equal 6 m.

The problems of unitizing the construction are most simply solved by replacing the overhead travelling cranes with outdoor lift-transport equipment, not connected with support structures, and by the location of the equipment on cleared and half-cleared ground and with the unitizing of structural designs.

The utilization of unitized structures is most feasible for one-story mine construction as for example, decknead buildings with haulage at the design level. Such deckhead buildings, without considering the presence of complex joints of the shafts, are similar to industrial shops. In this instance the utilization not only of unitized flooring designs is possible, but also of utilized sectional columns, shoes, foundation and overhead crane girders and so on.

However, the utilization of utilized structures even in the simplest two-story deckhead buildings is not an easy task. Thus, for instance, in a two-story building difficulties arise, induced by the presence of the flooring at the level of the upper docking area, located at a height of 6-30 m above the planning level and characterized by large spans and by considerable dynamic loads. The typification of such buildings should primarily begin with the selection of standard spans, beams and flooring slabs, the spacing of the columns and the connecting of the upper story of the frame of the filled walls. The composition of the unitized structures of a given group of a construction complex can be subsequently enlarged, keeping in mind the utilization of the minimum quantity of the type and dimensions of the sectional parts. These approaches are also applicable to other structures, which have solutions coinciding with the solutions of industrial buildings. Belonging to this type of construction are several modular frameworks with ferroconcrete and combined multiple-story frames. As flooring for these housings, the utilization of sectional farroconcrete and prestressed structures (Fig. 42) is possible as a rule.

The ferroconcrete above grade sectional frame of a crushing mill (Fig. 43) can be combined with the monolithic underground part of the framework, sometimes embedded several tens of meters below the planning level. In this instance the above grade framework for the most part, can be made using unitized structures (slabs, beams or capping girders, wall fillings, of window lintels, of foundation beams, particularly overhead crane beams and others).

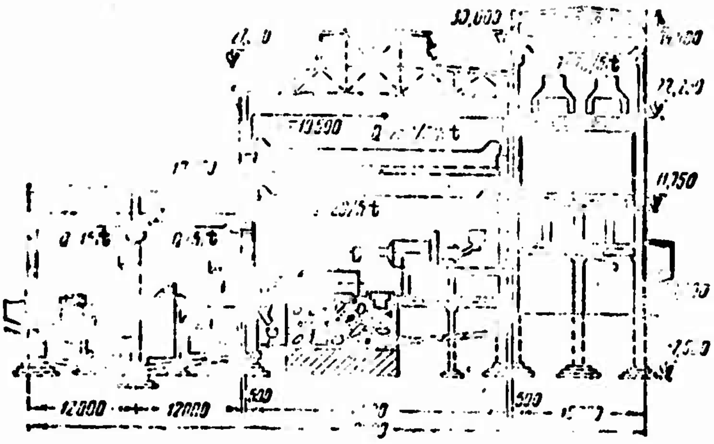

Fig. 42. A superstructure with preferred utilization of sectional ferroconcrete flooring and capping structures.

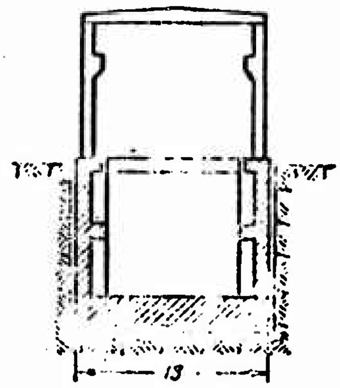

Fig. 43. Sectional above grade frame and underground monolithic part of the crushing mill housing.

As already noted, in mine buildings and related construction predominantly ferroconcrete support designs, especially sectional ferroconcrete columns, sectional prestressed overhead crane beams with spans of 6 and 12 m (with overhead cranes of a general purpose having a load-lifting capacity of 30 t), sectional ferroconcrete prestressed beams and flooring trusses and so on should be used. However, in a number of cases the complete utilization of sectional ferroconcrete in mine construction results in an excessively large quantity of typical dimensions and forms and in their slight use. On the other hand, a large amount of strengthening at the joints

of elements of structural frames also complicates the solutions of sectional designs. In most cases the installation of simplest metallic connections leads to a sharp decrease in the transverse forces and in the values of the bending moments in the uprights and joints of the frames, and to a substantial simplification of the parts and joints of the design. The reduction of expenditure of the metal achieved as a result of the utilization of metallic connections usually compensates for the expenditure of metal at the connection. However, the expediency of this measure should be examined separately in each individual case. Figure 44 gives diagrams of reinforced concrete frames. In Fig. 45 - the diagram of a multistage frame.

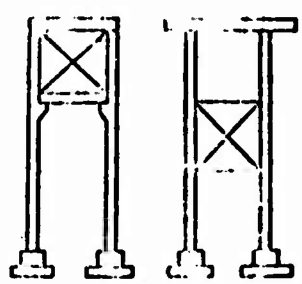

Fig. 44. Diagrams of ferroconcrete frames.

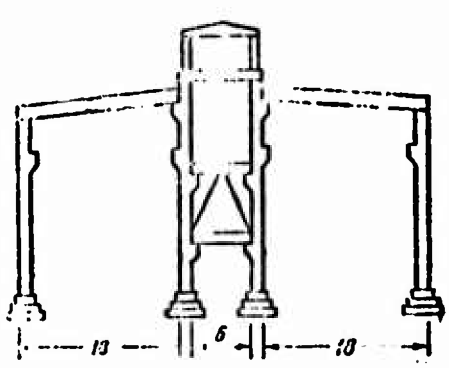

Fig. 45. Multistage ferroconcrete frame with metallic joints in the plane of the frame.

In a number of cases is expedient the partial utilization of metal in light superstructures which caps parts of the construction, skylights, individual spans. Figure 46 shows a diagram of a section of a sectional ferroconcrete loading ramp, made from repeating sectional elements (upright, cross-bar, main beam). This result was possible because of the introduction of a single nonstandard span of a ramp made of metallic span structure 1.

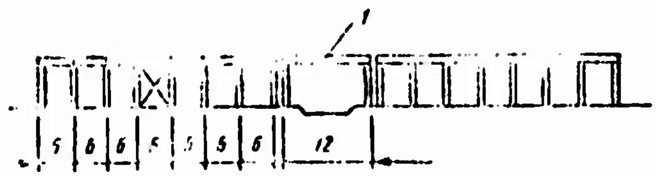

Fig. 46. A section of a ferroconcrete ramp with one metallic span structure.

Combinations of reinforced concrete frames in the loading hoppers and in other hoppers, where the container part of the structure is made of light metallic structures utilized for the transmission of expanding forces are quite completely expedient. These designs are described in Chapter XI.

In increasingly more or less complex structures the utilization of reinforced concrete is usually effective in conjunction with metallic structures. An example of this are mine heads with idler pulleys. The erection of a ferroconcrete mine head frame is complex and critical work whereas the manufacture and installation of the metallic mine head usually do not cause difficulties.

Individual mine structures can be made completely out of metallic designs. For example, the mine head with idler pulleys are usually made of metal. Furthermore, technical conditions permitting, there can be metallic designs for the bridges of the conveyor tunnels with spans of more than 18 m, supports for tunnels having a height of more than 14 m, deckhead buildings in sections close to the shafts, and also for flooring of deckhead and other mine buildings and related structures with a spacing of the uprights of more than 12 m, with spans of more than 30 (24) m. The utilization of the metallic designs for the floorings of deckhead buildings, which differ in complexity and in the response to considerable dynamic loads, one should connect the ferroconcrete sectional plates using spans of 3 and 6 m, which bear the ballast and traffic. Under these conditions the utilization of the metallic structures is expedient with a spacing of the uprights at 12 m or more.

The utilization of metallic columns and framework in industrial buildings involves great heights and large loads under heavy working conditions of the cranes. Figure 47 shows a diagram of the frame of a heavy modular superstructure whose composition includes parabolic kettle feed bins, ore storage bins and others. The framework is equipped with several overhead cranes, whose load-lifting capacity attains 250 t. The length of the framework is 756 m, and for its erection about 40 thousand tons of metal structures is necessary.

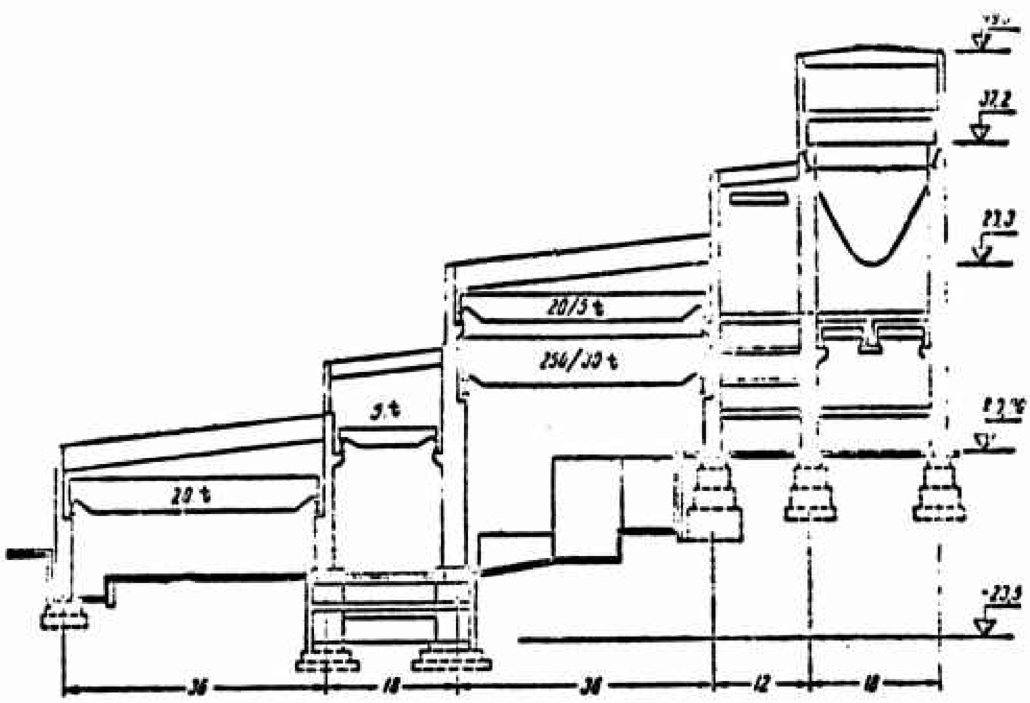

Fig. 47. The framework of an ore dressing plant.

For frameworks similar to that given in Fig. 47, just as for that noted above, the utilization of the combination of reinforced concrete designs with the maximally possible introduction of sectional ferroconcrete and preliminarily prestressed structures and parts is expedient: slabs and flooring panels, coverings, walls, channels, support walls, funnels, beams, hoppers; beams and trusses of the capping and based on feasibility, of span of frames, uprights and others. For a lesser number of frames based on size (see Fig. 42) the partial utilization of sectional ferroconcrete and prestressed designs of the frame has already been carried out.

The introduction of stressed metallic designs is expedient in conveyor tunnels in ore bins with the spans of the tunnels being 50-100 m, and also in some other structures. This leads to a decrease in the height of the trusses, to the possibility of placing the trusses predominantly according to the layout of the tunnels and to a reduction in the expenditure of metal. Under these conditions the suspended and composite diagrams of span structures in the construction of ore bins are possible.

For a number of pits the wooden frames used predominantly for cold construction are rather common. For protection in case of dampness, sanitation and the presence of natural ventilation, the utilization of simple and light wooden frames is expedient for a number of structures and it is completely effective over short periods when operating the mine. Figure 48 shows a diagram of a light wooden flooring frame of the tunnel work. Such designs with spans of 3 to 15 m are widely used in pits 30 years old for hauling, surface hopper, and conveyor tunnels and other structures. Frequently, these frameworks are set in ferroconcrete uprights, faced with asbestos-cement sheets which increases the degree of fireproofing and insures the covering of the wooden designs.

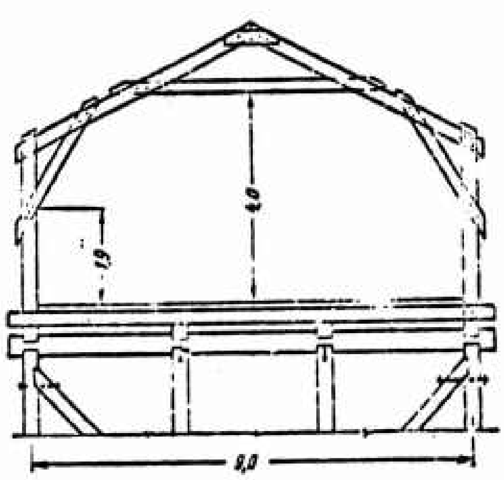

Fig. 48. Light wooden flooring frame of the galleries.

The wooden frames, reinforced with metallic structures, can be successfully used over a corresponding period of service of the structure in mine heads, wooden and wooden-ferroconcrete hoppers, and deckhead buildings. In the case of considerable loads on floorings, metallic cross beams of frames of combined design (Fig. 49) have been used with success.

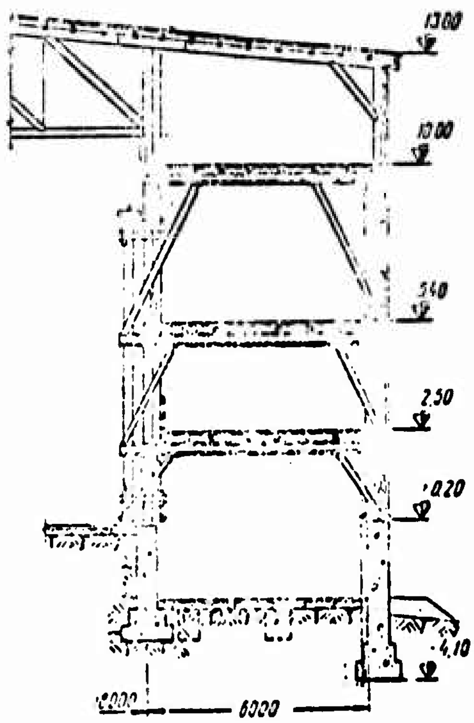

Fig. 49. The structural frame with reinforced cross beams.

2. The Elements of Construction

Foundations. For individual small construction including buildings, which have a relatively complex form by design, *continuous footings are used* - monolithic and sectional. The transverse profile of a continuous footing, as a rule, should be close to the form of a double-T, with the development in the base and towards the top - under the pedestal part of the wall. The width of the foundation in the middle part of the profile is taken as minimum: from a concrete 300 mm or more; from a rubble concrete, concrete blocks and stones of a proper form, 400 mm or more; from rubble masonry, 500 mm or more.

The development of the profile at the base upwards is done with ledges, depending on the material of the foundation. The ratio of the height and deviation of a ledge is usually taken fro rubble masonry, rubble concrete and concrete, from 2 to 1.2; for concrete this ratio can be reduced to 1. The development of a profile at the base is carried out mainly because of the introduction of sectional or monolithic ferroconcrete pad, 1-3 m wide at a height of 0.3-0.6 m.

In the actual construction of mines are most widespread columnar foundations of the inverted tumbler type. In connection with this is the fact that a mine construction in most cases has a complex underground makeup, which is characterized by the presence of tunnels, passageways, chambers, sumps, complex and sizable retaining walls, foundations for equipment, more expedient utilization of the foundations having the location of their top at the 0.05-0.15 m mark, i.e., approximately at the planning level. A similar arrangement of the foundations should be completed primarily having in mind the difficulties of moving construction cranes. In this instance it is necessary to fill in the excavated areas with soil immediately after installing the foundations; this facilitates the creation of the most convenient conditions for the movement of construction equipment. The edges of the foundations of the columns in this case are not made less than 300 mm; the usual minimum width of the upper step of the foundation is 1 m. The foundation beams rest on bosses, but with a width of the upper step of the foundation of more than 1 m, the beams are placed in niches of the foundations, flush with their top. A certain increase in the volumes of the foundations using these structural techniques is partially compensated for by the decrease in the length of the columns and of the foundation beams. The expediency of the raised position of the column foundations is lowered with an increase in the spans and with the reduction in the volume of underground activity, and also with the increase in the depth of laying the foundations. Under these conditions the utilization of elongated columns is possible at a lowered position of the top of the foundations relative to the planning level.

With the placing of the structure on old mine dumps, freshly compacted dumps, settled earth, slightly and relatively intensively consolidated soils and in other cases foundation ferroconcrete slabs have been developed by design and used with success. The utilization of the latter frequently allows one to avoid complex artificial bases, deep foundations, piles and caissons. Under mine conditions the presence of the large foundation slabs in most cases prevents substantial deformations in the construction with partially settled earth in the zone of direct abutting to the shaft, loading funnels and under other conditions.

In the hauling galleries foundation slabs having an area of 100 m^2 are used, and in the deckhead and other buildings - up to 1000 m^2. The diagrams of foundation slabs are given in division III.

With the infeasibility or undesirability of introducing settlement (shrinkage) joints and a considerable build up of slabs by design one ought to consider the uneven distribution of the pressures on the soil and the possibility of local settlement of foundation slabs with a perceptible (sometimes temporary) redistribution of the reinforcements in the slab. In such cases it is necessary to specify slabs appropriate to the steel framework.

Slabs of increased thickness and massive ferroconcrete foundations are application for various equipment, lift machines, crushers and others. Figure 50 given a two-level foundation, designed for the installation of two crushers. The height of similar foundations attains 20 m and more.

With sunken massive housing foundation slabs massive foundations, in a number of cases, serve as the foundations of the structure and equipment. Examples of massive foundation slabs, of supporting loads from the equipment and of the frame of crushing mill, are shown in Figs. 43 and 58. The greatest massive foundation is represented in Fig. 51. This foundation is designed for a group of highly productive crushers, and the structures of the framework are supported on it.

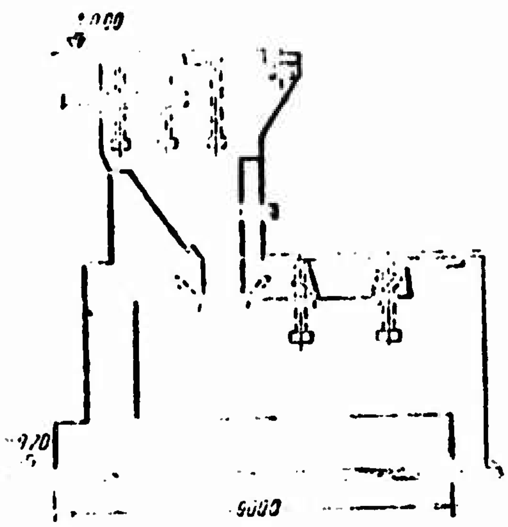

Fig. 50. A two-level foundation of a crushing mill.

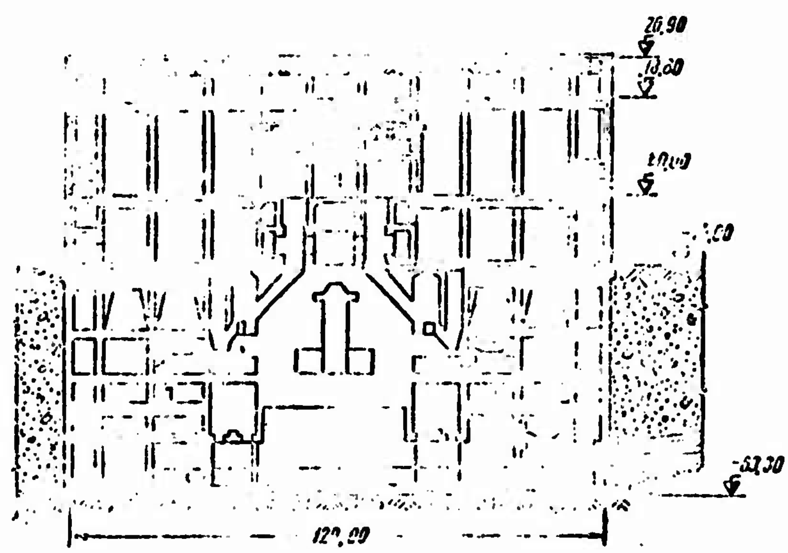

Fig. 51. The greatest massive foundation of a sunken mine housing frame.

The driven piles have limited utilization in connection with the soils, piles and dumps of rock inclusions and boulders, which prevent the driving of the piles. Furthermore, the level of the ground water under the conditions of mining, frequently fluctuates within considerable limits, and within thicker piles the ground water is usually absent, which precludes the utilization of wooden piles. For these reasons piles in a continuous section installed using scourings and vibrations, are rearely applied. Tubular ferroconcrete piles of a large diameter and piles in place of all types should be more extensively used.

Caissons are used in the construction of mines fairly often. The forms of the caissons by design can vary: round, hexahedral, polyhedral, square, square with mitered angles, rectangular and others. The selection of the form of caissons is done depending on the outline and mutual location of the structures and their foundations.

The depth of the caisson usually constitutes 10-20 m and attains 40 m.

The sizes of the caissons by design attain 40 × 40 m and more. Frequently they are not only used as foundations of buildings and related construction, but also as foundations for scientific equipment. The walls of one such caisson has a thickness of 1 m towards the top and 1.85 m in the lower part of the structure, reinforced with vertical strengthened frames made of 8 rods of a periodic profile 36п, set at 1.6 m from the perimeter. With a diameter of a caisson at 33 m and a height of 27 m the expenditure of concrete constitutes 4250 m^3, the steel reinforcement, 470 t.

Figure 52 shows a caisson with a diameter of 43.7 m at a height (to a depth) of 29.3 m. The walls of this caisson have a thickness of 1.5 m towards the top and 2.35 m in the lower part. During the construction of the caisson 478 t of reinforcement frame was erected, 6217 m^3 concrete was poured, 37 thousand m^3 of soil excavated. For the earthen work two excavators with scoops having a capacity of 0.5 m^3, and a bulldozer, were used. Soil was lifted to the surface by tower gantry cranes in metallic dumping bodies with a capacity of 2 m^3.

The location of the foundations on permafrost soils is not seen as a common place phenomenon with respect to deckhead buildings and other structures, but is seen at mines in the north and north-east regions of the USSR. Under these conditions it is necessary to use the experience of erecting various buildings and related construction on permafrost soils. The given problem is quite specialized, associated with problems of permafrost studies and cannot be presented in the present chapter.

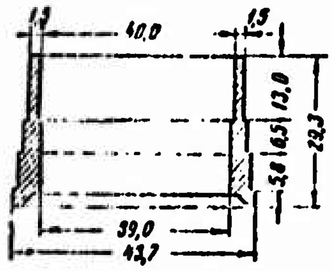

Fig. 52. A cylindrical caisson with a diameter of 43.7 m.

In a number of cases the installation of the construction and their foundations on permafrost soils is done using the principle of preserving the permafrost. During the operation of the structure negative temperatures of soils of the building site are maintained which are most simply carried out in unheated construction, ramps, galleries, bridges, various storage bins. In heated buildings and related construction one focuses his attention on the equipment of the cold underground, its reliable ventilation, the preservation of conservation low temperatures here. In this case as well as others the supports of the structure will sink into the soil by a calculation of their reliable anchorage, which prevents the development of the swelling forces of the active layer of the soils. Figure 53 gives an example of construction, which is elevated based on the principle of preserving the permafrost.

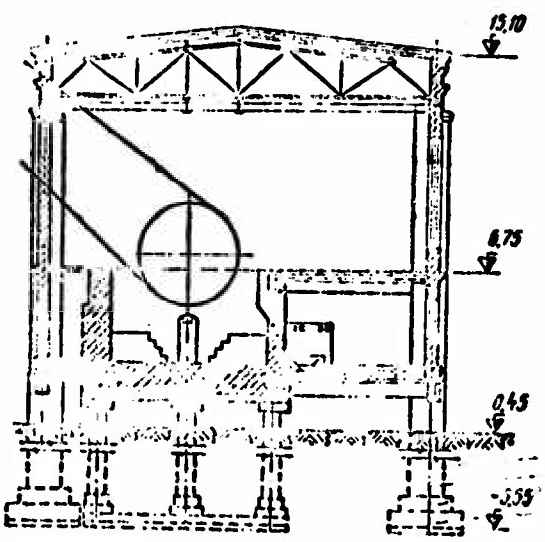

Fig. 53. Building for lift machines, designed on the principle of the preservation of the permafrost.

The degree of complexity of laying the foundations increases with their concentration on relatively smaller sites, especially, mine shafts, where a pile driver and other surface construction, hauling galleries, lift-loading hoppers, receiving funnels of crushing-grading devices, of factories and of ore-dressing combines, ducts for the supply of air, passable and impassable tunnels, foundations for equipment, and others are usually concentrated. All the enumerated structures are organized on a small site, as a consequence of which the arrangement of the numerous foundations is hampered, and the individual foundation units are divided sometimes by only seams.

Figure 54a gives an example of a unit of foundations for heavy iron-ore shafts, where on a small section having an area of about 1000 m^2 a series of foundations of mine construction is concentrated. The presence and development of the foundations are determined as follows.

The shoring of the mouth of the mine shaft 1 is done by a pile driver and has a series of devices, niches, apertures. Caissons 2 with an inscribed diameter of about 8 m at a depth of 16 m transmit about two thousand tons of various loads to the soils of the foundation from the lift-loading skip bunkers. The transmission of the loads from the hoppers to the soils located above in this instance was not possible. The foundations of the deckhead and related construction - hauling galleries 3 for a given unit are substantially developed in connection with the need for the purpose of spans of galleries within the limits up to 30 m. Unlike the foundations, fill soils have been accepted as a base of a deckhouse building. The ferroconcrete receiving funnel 5 abutting against the unit of the crushing-dressing mill is also sunken considerably lower than the level of the surface relief; the foundations of the funnel are lain at a depth of 16-18 m.

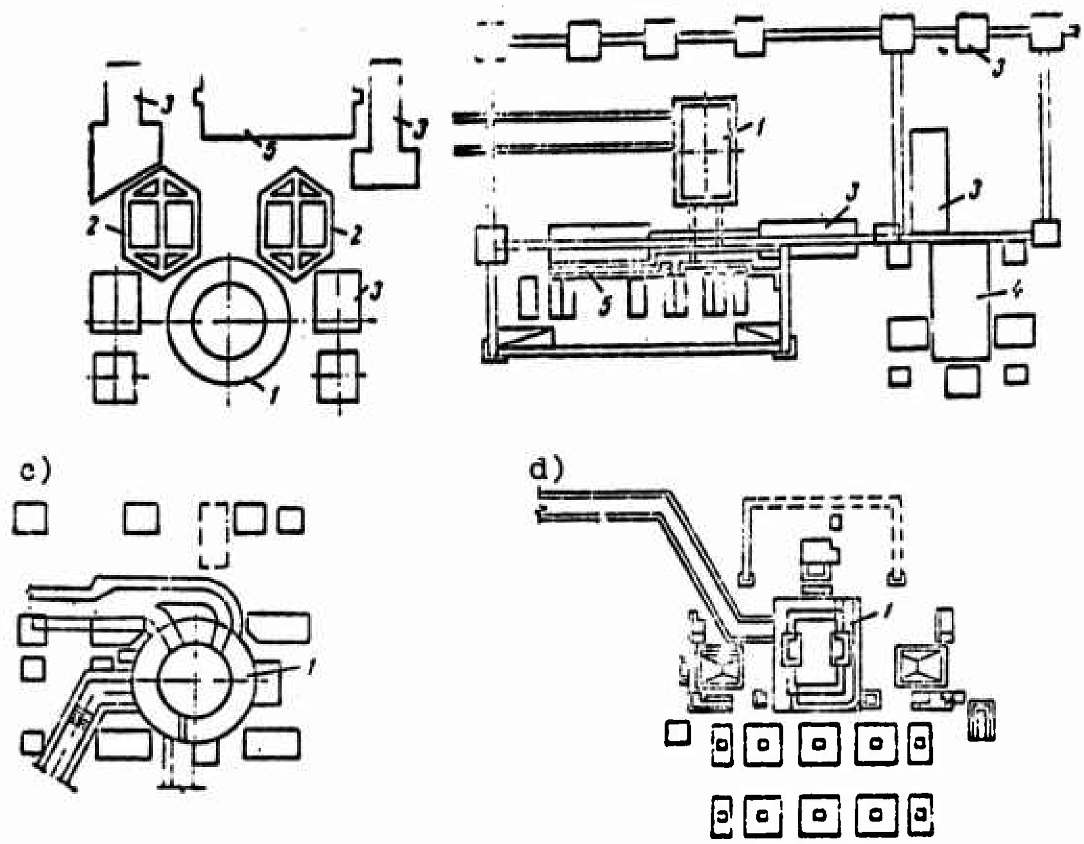

Fig. 54. The units of the foundations, positioned abutting against mine shafts: 1 shoring - mouth; 2 - caissons - the artificial foundations of the hoppers; 3 - the foundations of hauling galleries and of a deckhouse building; 4 - the foundation of a crushing mill; 5 - the receiving funnel of the crushing-dressing mill.

Figure 54b depicts a unit of the foundation for a small iron-ore mine shaft. Figure 54c shows the unit of foundations for the shaft of one additional heavy iron-ore mine. Figure 54d shows the unit of the foundations for a shaft of one of the heavy mines of the copper-ore industry.

Depending on the distance from the shaft the compactness of the foundation sites of the construction, their quantity and sizes are usually reduced. However, even on these sites, the various structures, for example hauling galleries, have considerable height and have greater extent, which attains 500 m at corresponding values of the areas, and also they are characterized by large loads and by built-up foundations.

Enclosing elements and temperature-moisture conditions of their operation. At industrial mine sites lacking ventilation or having unsatisfactory operation the working conditions with respect to temperature and moisture frequently act unfavorably as barriers. In enclosed storage bins of moist ores and concentrates, deckhouse construction (buildings), at many industrial sites with the washing of the floors, including the galleries, surface and underground hopper sites, sorting equipment, the separation areas, washing devices and plants and related housing having a moisture source, the humidity, as a rule, has increased.

In a number of cases at industrial sites unfavorable temperature-moisture conditions are created separately. For example, the condition at a site, where the ore is located following a process of drying. The moisture of the ore which is passed through the drying drums, amounts to about 15% at a temperature up to 80°. The dried steaming ore enters the conveyor galleries, where as a result of the small volumes of the working space and the absence of effective ventilation the relative humidity attains 90-100% at a temperature of 15-25°. One observes analogous and worst conditions in galleries and adjacent construction with the so-called return cycle of agglomeration plants, where the relative humidity attains 90-100% at a temperature of 25°; also, at sites for the thawing of an ore, where the relative humidity constitutes 100%, and at a temperature - 100°.

In all shown cases and in other such cases ventilation equipment is necessary. However, the complexes of construction and buildings of mining enterprises are characterized in a number of cases by a large number of industrial sites, located at various levels, in numerous galleries, at transfer units and at other points. On the other hand, a rather high permissible limit of relative humidity (80%) and a temperature (23°) which act as standards in the working zone of the industrial sites, close to the above shown indexes has been established. Therefore, in a number of cases during the examination of the designs for barriers, one ought to take into consideration along with the practical indexes, the values of the temperatures and of the relative humidity of the industrial mine sites. In this case

it is necessary to consider that the humidity and temperature at the sites are relatively constant over the course of twenty-four hours. Applicable to these indexes it is possible to preliminarily differentiate the sites into separate groups.

Thus, for a group with a temperature below 15° and a relative humidity of less than 60% one can include the sites of open hauling galleries, crushing-sorting mills and factories with conveyor galleries (during the dry dressing process), lift machines sites, maintenance workshops, dump truck stands, the depot for dump cars and travelling machines.

One ought to consider separately the sites of multirope lift machines. The temperature of the air at these sites can reach 20° and, to a considerable degree, depends upon the effectiveness of the artificial ventilation.

For a group with a temperature below 15° and a relative humidity of 61-75% one can include a number of deckhouse structures (deckhouse buildings) and enclosed hauling galleries abutting to mine shafts; the crushing and sorting mill sites, surface and underground hopper sites, transfer unit sites and conveyor galleries equipped with washing floors and walls; washing equipment, wet dressing plants, the enclosed storage bins for moist ores and concentrates.

For the group with a temperature below 25° and with a relative humidity of more than 75% one includes the galleries of the return cycle, the galleries of ore drying housing, utilized for stockpiling slightly dried ore, and the separate sites of plants handling wet dressed ore. Flushing abrasive and other wet sites of daily operated enterprises are characterized, furthermore, by an increased risen temperature of those sites, climbing up to 25-30°.

Industrial sites for the defrosting (thawing) of an ore, where the temperature comprises about 100° at a relative humidity of 100% constitutes a special group.

Floorings and cappings of mine construction are usually done in the form of dense and steam-tight, moisture- and frost-resistant ferroconcrete designs while for the walls one still uses brick masonry, masonry from warm concrete blocks, slag blocks and other kinds of material unstable under conditions of increased humidity. Therefore, the choice of wall barriers for mine construction in all cases deserves special attention.

The problems, connected with research of the material of the walls (and other enclosure designs) in the humid state, in general, are very complex as a result of a considerable number of factors, which influence the moisture content in this or some other section of the barrier. Included among these factors is climate (temperature and humidity) at various times of the year, velocity and direction of the winds, atmospheric precipitation, various combinations of the listed factors), temperature-moisture condition of the air at the sites, the initial humidity and physical properties of the structural material (the indexes of sorption humidification, the data about the capillary phenomena, and others).

The process of humidification of a barrier is considerably accelerated in the presence of moisture on the inner surface of the walls, which formed during the condensing of vapors, the washing of walls and so on. A part of this moisture is absorbed by the material of the barrier and is moved into the outer layers of the walls.

Parallel to the movement of the moisture, during a cold spell the movement of the water vapor also occurs, which penetrates through the barrier as a result of the difference in the vapor pressure in the interior and external air. The limiting vapor pressure E in mm Hg at certain temperatures, $t°$, and at normal barometric pressure constitutes:

$t°$	−40	−30	−20	−10	−5	0	+5	+10	+15	+20	+25	+35
E	0. [illegible]	0,28	0.77	1,95	3,01	4,58	6,54	9,21	12,79	17,54	23,76	42,18

During the cold period of the year the vapor pressure in the external air is insignificant and is also frequently close to zero. Therefore, the process of the movement of water vapor through a barrier is determined basically by the absolute humidity at the site.

The absolute humidity of the sites (at +18°) within the limits of 8-9.9 mm Hg (relative humidity within the limits of 50-60%) is considered that which corresponds to the normal condition, an absolute humidity of 10-12.5 mm Hg (61-75%) is considered that which corresponds to himid conditions, and over 12.5 mg Hg - to wet conditions.

The high humidity at the sites leads to the humidification of the barriers, to the failure of the external layers of the walls under the pressure of the moisture and frost. The need to give special attention to the shown questions is confirmed by the numerous examples of failure of wall barriers made from brick and other masonry as related to Russian and foreign, and especially, Swedish experience in the practice of mine construction operations.

Along the unprotected walls at moist and wet sites outer plastering, which scales and falls is destroyed, having been absorbed as a part of the masonry. In the absence of plastering, failure of the masonry facing shows up primarily in the least durable sections of its outer surface, which with height of one or of several courses of brick it partially crumbles; moreover, the most intensive failures are observed in the sections having the least thickness, i.e., where there is least resistance to the permeation of water vapor near various cracks and seams, in window stool sections, along the outline of the flooring abutments, especially in the supporting sections of sectional trusses and others.

With further humidification and thawing the masonry facing is is destroyed to a depth of up to 250 mm and more from the outer side of the walls. The process of scaling of the damaged masonry in some places is so sharply expressed, that the separation of the external layers of the wall in them does not suggest a specific action. During a significant development of the process of failure of the walls, their

crumbling with a loss of the facing is possible and in this case with a certain heaving action on the walls leads to an even greater magnitude of failure.

In order to eliminate the possibility of such failures it is necessary to take strong measures, directed towards the exclusion of humidity and eventual failure of the walls of mine buildings and related construction.

The designs of wall barriers should be initiated in accordance with the requirements of the structural heat engineer. Briefly given below are merely the corresponding chief checks.

The first check amounts to a determination of the degree of resistance to heat transfer R_0, which should be more than the required value, R_0^{TP}, initially recommended according to the structural standards. The degree of resistance to heat transfer diminished in comparison with the normal value, R_0^{TP}, leads to the possibility of the condensation of a considerable quantity of moisture on the interior surfaces of the walls.

During the second check the degree of resistance of a barrier to vapor penetration is determined

$$R_{\text{п}} = \Sigma \frac{\delta}{\mu},$$

where δ - the thickness of the layer, m; μ - the coefficient of vapor penetration of the material, g/m, mm Hg, part 1, derived according to tables and equal to 0.004 for reinforced concrete; 0.01-0.02 - for cement and complex motars, slag concrete, brick laying, lime plastering; 0.01-0.03 - for cellular concretes; 0.06-0.065 - for mineralized felt. The value R_{Π} increases by 4.8-3.6 m^2 mm Hg parts/gram with thorough oil or enamel staining of the surface of the barrier.

The resistance of vapor penetration of the inner half of the thickness of the external wall sites with an absolute humidity is

more than 9 mm Hg should be more than the resistance of the external revetment, or more than

$$R_{u}^{np} = 2{,}4\,(e_{в} - 9)\,K,$$

where e_{B} - the vapor pressure of the outer air, mm Hg; K - coefficient equal to 0.7 for a zone with a climate of moderate humidity and 0.9 - with a moist climate.

During the check of possible condensation of water vapor in various sections of the wall barrier it is convenient to use the method indicated below, based upon the assumption of constant flow of water vapor through the barrier. Such a flow is established over weeks and months after the formation of a determined values of vapor pressure. Therefore, the checks of the possible condensation of vapor pressure inside the wall are made predominantly for multilayer designs, where these checks serve as a comparison of diverse forms of wall barriers. It is considered that such checks as applied to massive uniform barriers have an arbitrary character, and one determines maximally the possible humidity of the materials of the barrier. However, as was already noted, temperature-moisture condition of the site in the mine buildings and related construction is practically constant which serves as a base for the comparatively broad utilization of the shown checks, which amount to the following.

The temperatures are found on the interior surface of the barrier τ_{B} and the interior surface of any layer τ_{n}, and the values of the limiting vapor pressures E for temperature τ_{B} and τ_{n} are determined.

For the same interior surfaces of the layers of a barrier one finds the vapor pressures e_{n}, determined for the expression

$$e_n = e_{в} - \frac{\Delta_e}{R_n} \Sigma r_{n\,(n-1)},$$

where e_{B} - the vapor pressure in the air at the site; Δ_{e} - the difference in the vapor pressures in the inside and outside air;

R_{Π} - the overall resistance of the vapor penetration of all layers of the barrier; $\Sigma r_{\Pi(n-1)}$ - the resistance of the vapor penetration of the interior layers, located between the site and the plane where the value of the vapor pressure is calculated.

In those layers where the value e_n is less than the values, E, the condensation of vapor in the mass of the wall is disregarded; on the other hand, the condensation of vapor is possible in those sections of a wall where the values e_n are higher than the values of E (determined for the same vertical sections parallel to the layers of the wall).

As an example a check was made of the possibility of the condensation of water vapor in sections of the wall panels, composed of support ferroconcrete slabs with a minimum thickness of 30 mm, with a layer of mineralized felt having a thickness of 80 mm and an overall protection with slabs having a thickness of 15 mm. The joints of the support slabs are thoroughly compacted, whereupon a resistant coating is applied to the interior surface of the panels. The joints of the protective slabs are partially consolidated.

The temperature of the site is +15°, the relative humidity is 80%, the external temperature (the mean for the coldest month) is -20°; the relative humidity of the external air is 78%.

The results of the check are presented in Table 7.

The data in Table 7 were derived for the design of the wall panels with dense support (interior) ferroconcrete slabs. The check, made on a brick wall having a thickness of two bricks, found under the same conditions as that for a wall in the example examined here, gives a negative result. Such a result corresponds to indications of active structural standards. According to the standards, based on the interior surface of the external walls of moist and wet sites a device is necessary of a protective-screening vapor-insulated layer, made of moisture-proof material and which amounts to in a number of cases, to a device for the continuous vapor-insulated, protected concrete or brick screen and so on.

Table 7.

Layers	Vapor pressure e, mm Hg	Limiting vapor pressure E mm Hg	Conclusions
Interior surface of the wall	10.2	10.4	Possibility of condensation of water vapor disregarded
Junction of support slabs with heat insulation	1.7	10.0	The same
Middle of the layer of heat insulation	1.3	5.5	The same
Interior surface of the protecting slabs	0.9	0.9	Possibility of condensation is practically disregarded
External surface of the wall	0.6	0.87	The same

The walls of a considerable number of mine structures should be protected with a vapor-insulated layer. In practice, however, this requirement frequently is not met partly for reasons of the lack of data about the actual air humidity of industrial mine sites, and also as a result of the complexity and high cost of brick barriers with vapor insulation. Wide utilization of efficient vapor insulation in a number of cases is debatable; its actual use on a large scale is highly improbable.

Walls made from brick and other unprotected masonry under conditions of mine building in a number of cases under humid conditions are not a rational solution of the barriers. According to the data of observations, conducted in Sweden, valid for the north and middle zones of the USSR, the walls from unprotected brick masonry and warm concrete should not be used in the composition of deckhouse structures, of surface pile drivers, ore-dressing and flotation plates and others.

Wall, flooring and cappings. The walls, flooring and other elements of mine structures undergo the pressure of explosions, exist under unfavorable temperature-moisture conditions, bear various dynamic loads, which appear as a result of the work of lift and power equipment, the unloading of lift containers and so forth.

According to industrial conditions the walls, flooring and cappings of structures should be fire-proof. Simultaneously, in connection with the limited period of the operational goals these

elements of construction should have small weight, should be sectional, convenient to install and move to adjacent sites. The last circumstance has special value when exploiting groups of mutually adjacent systematically worked deposits. In these cases the repeated utilization of the elements of structures can substantially affect the technical-economical indexes of the undertaking and to determine the expediency exploiting certain deposits. Independent of the shown particular case, the secondary utilization of repeating [build-on] units (primarily the panels of walls and flooring, standard beams) always expediently results in considerable reduction in the cost of building.

Recommendations bases on the selection of the elements of the structures of walls, floorings and cappings amount to the following. In mine construction there should exist the possibility of avoiding the utilization of heavy, massive designs of support stone walls. It is recommended to limit the utilization of such walls to a small group of relatively small complex construction and buildings based on sizes and by design (short surface galleries and the load-transfer points form by desing, small winch buildings and others). Stone, brick and other masonry for a wall filling in the frames of extensive deckhouse buildings (construction), in the frames of hauling and conveyor galleries should be excluded if at all possible. During the erection of the masonry of all types, as a rule, antiseismic measures are carried out.

The utilization of stone masonry in self-supporting walls of considerable length and height of the latter, is not recommended. The sections and the weight of self-supporting stone walls in such cases are made very large, which is governed by the emergence of considerable seismic forces. These walls should have the required number of antiseismic straps connected to the building frames, and to other elements. The thawing out of even the separate sections of self-supporting walls can lead to damage.

The utilization of stone, including brick masonry in two-layered and multilayered designs of wall barriers is not recommended.

The filling of the walls based on the Gerard type masonry which consists of two brick walls having a thickness of 120 mm, between which there is heat insulation, is inadmissible under conditions of blasting. Similar and wall fillings composed of two layers of brick masonry in combination with warm concrete and other material are also inadmissible. Two-layered wall barriers, composed of lightened brick masonry (including brick masonry having a thickness of 120 mm under a metallic frame), warmed by slabs of cellular concrete which are used sometimes, cannot be recommended under conditions of blasting.

Walls made of heavy units of considerable volumetric weight based on earthquake-proofness are equivalent to support walls made from stone masonry, and in general, are not a satisfactory design for construction, located in the regions of blasting, and cannot be recommended for broad utilization under these conditions, despite the known advantages of masonry made of heavy units. It is recommended to apply such masonry for the walls of individual warehouse and industrial buildings, when the utilization of the wall panels for any reasons is contraindicated.

Masonry made of heavy units of a small volumetric weight is characterized by substantially less seismic loads and therefore is, to a greater extent, applicable for buildings under conditions of blasting, that is to say, in the case when heavy units satisfy the requirements of high strength and frost resistance.

Heavy wall panels are the most expedient filling for walls of mine construction. However, the majority of the known panels are made without taking into account the special requirements of mine building and operation of construction. Therefore, it is necessary to pay attention to the selection of the existing designs of wall panels. Thus, for instance, in general it is not possible to recommend large-size reinforced foam concrete slabs at mine sites for broad utilization because under conditions of increased humidity at the site there is the danger of the corrosion of the steel framework

and possible thawing of the walls. Under various unfavorable conditions of operation of mine construction wall panels are necessary which can well resist the action of low temperatures and moisture, explosions, vibration during the operation of the equipment, and so on.

The possible schemes of employing wall panels of warm industrial buildings and construction, located in areas of blasting, are given in Fig. 55. According to the first and second schemes (Fig. 55a, b) a three-layered panel is formed by two heavy ferroconcrete slabs with ribs, turned to the inside of the wall and mutually connected by means of welding the matching parts; between the slabs effective heat insulation is placed. These known designs, proposed by Engineer G. I. Potapov and his co-workers, recently replaced the design of vibration-rolled panels with the location of heat insulation between the relatively dense network of ribs of the ferroconcrete slabs. The panels in the second scheme (Fig. 55b), used in the housing of power stations (and also in residential houses), differ by the complete filling of the interior space of a panel with heat insulation, by predominantly mineral felt.

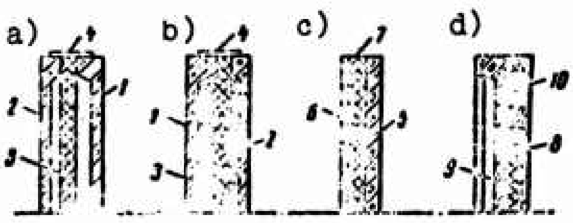

Fig. 55. Wall panels: a, b, c, d) the diagrams of wall panels of various designs; 1 and 2 - ferroconcrete ribbed slabs; 3 - heat insulation; 4 - the welding point of slabs with the aid of matching parts and facings; 5 - a support prestressed ferroconcrete slab; 6 - light facing slabs made from armoured cement, asbestos-cement or lightened ferroconcrete slab; 7 and 8 - heat insulation; 9 - ribbed support ferroconcrete slabs; 10 - the facing slabs.

One ought to indicate that the panels in the first and second schemes are made of ferroconcrete slabs of identical thickness and of uniform strength which results in an over-expenditure of material of the support designs, and according to the second scheme (Fig. 55b), furthermore, it results in the unjustified large expense of insulation

material. Panels 6 m in length according to the second scheme are characterized by a given thickness of reinforced concrete of about 0.10-0.12 m and a mineral fiber of 0.18 m with the weight of the wall filling of 300-350 kg/m^2.

Given in Fig. 55c, d are the schemes of panels of industrial buildings with support and flooring slabs which do not have these indicated deficiencies. The design of the wall panels according to the third scheme (Fig. 55c) consists of a support flat prestressed slab 5, a layer of heat insulation 7 and a light flooring slab 6, made of reinforced cement, asbestos-cement or made in the form of a lightened ferroconcrete slab. The design of the wall panels according to the fourth scheme (Fig. 55d) consists of a ribbed ferroconcrete carrying slab 9, whose ribs are directed from the wall, which results just as in the preceding case (Fig. 55c), to a minimum expenditure of insulation material.

The wall panels, shown in Fig. 55c, d, with the support and flooring slabs are characterized by small weight (on the average, 150-200 kg/m^2, of the wall) and by better (in comparison with other solutions) indexes of the expenditure of cement and insulation material. The given thickness of reinforced concrete and mineral fiber amounts to 0.05-0.07 m, but the cost is equal to 60-85% of the cost of the equidimensional thickness of two bricks based on the area of the wall. With the calculation of a number of other conditions of operation of mine projects three-layered panels and specifically, panels with support and flooring slabs are rational to a large degree for utilization in the walls of mine buildings and related construction.

The arrangement of the walls with the increased relative humidity, the sites where washing of the walls and ceilings occurs, and so on, were discussed previously. However, a number of the buildings at mine sites is characterized, relative to a favorable moisture condition, by the industrial site, that it does not exclude, for example, the utilization of panels made from reinforced cellular concrete. But the simultaneous utilization at a site of a small

shaft of two principle and different solutions of wall panels is inexpedient and usually is excluded based on the motives of unifying the designs. At large mines in the absence of several groups of construction and buildings with measures for moist and wet conditions, the utilization of single-layer panels made from reinforced cellular concrete, in a number of cases, is quite expedient. For the manufacture and interlocking of these panels one should meet the raised requirements. The grade of cellular concrete (prismatic strength) should not be less than 50 kg/cm^2 (with a cubic strength not lower than 100 kg/cm^2). For the thickness of the panels one ought to assign, based on a calculation, a certain excess of the required value of resistance to heat transfer ($R_0^{\text{тр}}$) by construction standards, determined for sites with the worst temperature-moisture conditions. On the inner surface of the panels it is expedient to apply a layer of dense motar so that the upper panels and planes of the walls on the whole should be waterproof (painted with enamels of the (ПХВ) [PKhV] polyvinyl chloride type or with other stable enamels and paints). Steel framework should be protected from corrosion by means of appropriate treatment, by increasing the protective effect of the concrete layer, and also by means of applying a hydrophobic composition on the external surface of the panel. The interlocking of the panels under unfavorable conditions of operation should exclude the possibility of damage with the failure of one anchor.

The utilization of large-size wall panels is expedient not only for warm, but also for nonheated construction. In this instance with the spacing of bars at 6 m, the weight of the panels constitutes on the average 140 kg/m^2 of the wall, which constitutes about 20% of the weight of the wall per $1\frac{1}{2}$ bricks. The cost of the large panels for wall of nonheated construction constitutes 50-70% of the cost with an equidimensional thickness per $1\frac{1}{2}$ bricks based on the area of the brick wall. With the acquisition of cold filling of wall of mine construction and related buildings it is also necessary to avoid the possibility of utilizing masonry made of brick and blocks for this purpose. Sone masonry can be allowed only for small separate units, with relatively complex outlines of the construction, nonstandard

sizes, reconstruction, and so forth. In all the remaining cases the walls of the cold buildings should be made of prefabricated ferro-concrete or stressed reinforced panels.

The longitudinal wall of the housing made from stone masonry is not a supporting one from the point of view of sustaining the loads from the large-panel slabs of the flooring. Therefore, at the level of the cornice a reinforced stone strap is usually installed, connected at intervals of the supporting structures with the flooring along with the help of rods of steel reinforcement placed between the slabs. With the filling of the walls with large panels, for example, with wall panels having support and facing slabs, the latter are braced to the uprights of the frames of industrial housing according to the height of the wall. In this instance the need arises for the employment of antiseismic belts. Designs of the corresponding cornice knots are shown in Figs. 56 and 57.

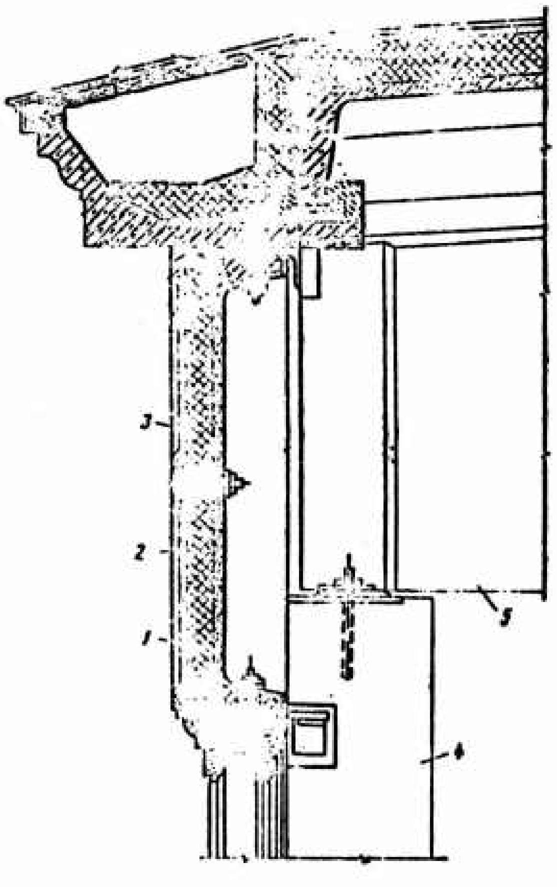

Fig. 56. The part of the wall filling of a mine structure with large wall panels along with support ferroconcrete and light facing slabs: 1 - support ribbed ferroconcrete slab with standard sizes, 6 × 1.2 m; 2 - light facing slabs; 3 - effective heat insulation; 4 - ferroconcrete column; 5 - structural flooring beam.

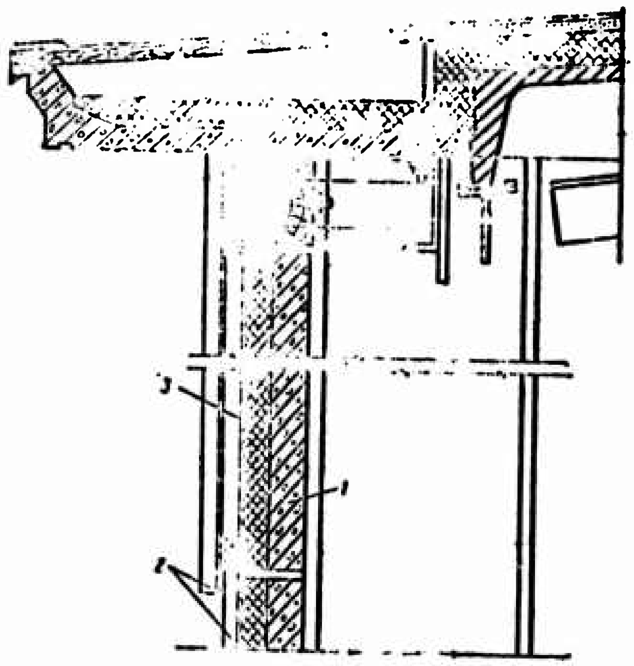

Fig. 57. The part of the wall filling of a tower pile driver frame with large wall panels along with supporting prestressed and light facing slabs: 1 - supporting prestressed slab with standard sizes, 6 × 1.2 and 9 × 1.2 m; 2 - light facing slabs; 3 - heat insulation.

In certain cases lightened wall fillings are used. As lining of the light nonheated galleries most frequently asbestos-cement wavy sheets are used. Warm wall fillings of this kind are made by means of arranging shields made of a light frame - the packing, made of effective heat insulation and lined on two sides with asbestos-cement wavy sheets of a strengthened profile. That described and other designs of the light wall filling are used predominantly in horizontal and inclined conveyor galleries of various spans.

For supporting designs of flooring the most frequently used are sectional ferroconcrete large-panel prestressed slabs 6 × 1.5(3) and 12 × 1.5(3) m (with light heat insulation over the slabs), welded slabs to beams or to trusses of the flooring and unitized slabs based on the entire area of the latter. Completely unitized flooring slabs are used under ordinary mine conditions (beyond the limits of the radius of the seismic dangerous zone) which consist of rather rigid discs - diaphragms. For the upper span pieces of the frames of mine construction and buildings, as a rule, are applied standardized

beams and flooring trusses, including stressed reinforced double-T beams, homogeneous beams, bench-made beam using steel framework made from high-strength wire and rod steel framework from low-alloy steel, single-rolled and double-rolled surface with spans of 12 and 18 m. In a number of cases stressed reinforced beams of double-T are used, assembled from blocks with steel framework made from high-strength wire beams with clamping devices in the form of pressed sleeves, single-rolled steel with spans of 12, 15, 18 m and double-rolled steel with spans of 12, 18 and 24 m; T-beams with conventional reinforcement, single-rolled steel with spans of 6 and 9 m and double-rolled steel with spans of 6, 9, sometimes 12 m; one should use wider stressed reinforced trusses for the flooring of constructed buildings, unitized trusses, and also assembled trusses made of semi-trusses and rods.

In a number of cases the utilization of ferroconcrete pre-stressed segmented trusses made from linear elements with spans of 18, 24 and 30 m is possible. The steel framework of these trusses in the first case is represented by high-strength wire of a periodic profile or by rods of a periodic profile, and in the second case - by beams made from smooth high-strength wire or by rods of a periodic profile.

With rather usual increased humidity at industrial sites under mine conditions one should use dense concretes for the manufacture of ferroconcrete trusses and beams and should protect the structure by means of painting them with stable varnish and paint compounds. With high humidity of the industrial site one should use only rod steel framework, dense concretes with retardation additives of steel corrosion (sodium nitrates and others) for the manufacture of ferro-concrete structures to provide the protection of the designs by means of painting them with stable varnish and paint compounds.

For the flooring of mine structures and buildings the utilization of standarized slab-flooring and span pieces of standard multi-story industrial buildings with various temporary loads on the flooring is sometimes possible. The utilization of these slabs and beams

is possible, especially, in conveyor and above ground hopper galleries, in warehouse and industrial buildings. This does not exclude their utilization in small deckhouse buildings with a skip lift and in some other cases which are characterized by floor loads within the limits up to 2 t/m^2. Frequently, however, special designs are necessary because temporary loads on the flooring in cellular deckhouse buildings, hauling galleries and ramps reach 6-12 t/m^2.

In deckhouse buildings are frequently used ferroconcrete slabs with spans of 3-6 m, main beams with a span of 6-24 m, sectional transverse frames with spans of 6-12 m along with stressed or conventional reinforcement slabs, main beams and spans of frames. The utilization of 6-meter ribbed flooring slabs, which bear directly on the span pieces of the frames of a deckhouse building is expedient.

The described solutions are also possible in hauling galleries. In hauling galleries it is frequently necessary to install a larger number of spans of main beams or of corresponding trusses along with the utilization of stressed reinforcement of the designs. The spans of the slabs remain usually equal to 3-6 m. The slabs rest on the main beams, and with the combined metal reinforced concrete measures for the galleries the laying of the slabs on the transverse beams is possible.

Sometimes a combination of slabs and main beams with a buildup of an upper strap of the latter is possible and the creation of a continuous series of beams based on the type of the upper structures of highway bridges is possible. This design for deckhouse buildings is practically excluded, and for hauling galleries it is a partial solution because it can only be applied when lacking the need for unloading the vessels within the confines of the galleries.

As it appears from that given, for flooring material of deckhouse and other structures special designs are necessary. Preferred here are combination and ferroconcrete designs with stressed and conventional reinforcement.

In a number of cases the utilization of standarized designs of ferroconcrete overhead crane and columns and those without them is possible. But more frequently the column of the construction differ substantially in their sizes and loads from the columns of industrial buildings. In connection with the presence of braked, impact, seismic and other horizontal loads, one should turn his undivided attention to the connecting of columns. The connections and in certain cases, the planes of the frames, as well, are attached to the uprights of the frames. The introduction of similar metallic connections is frequently expedient. Its application is usually relatively simple and its performance reliable in operation. Their utilization is usually accompanied by a minimum quantity of dimensional type of sectional ferroconcrete parts. In a number of cases one ought to pay special attention to the connection of the beams and main girders. The bracings in the supports of the main girders, beams, span pieces, trusses should exclude the possibility of displacements, shear blasting and other loads. As a rule, one ought to secure the joining of the beams with beams and columns by means of welding together the matching parts.

Floors and other elements of construction. The design of floors is made in accordance with the conditions of their operation. By conforming to industrial sites in the construction of a lift, it is possible to only apply certain designs of floor coverings.

In deckhouse buildings (construction) - with haulage - and in hauling galleries a floor covering of concrete, concrete slabs and blocks, slag castings, basaltic and other rock castings, clinker, and so on are expedient. In this case just at for the underlying layer, heat insulation, ballast, and material for the layout, most frequently slag is used. The floor covering in adjacent mine passages to a deckhouse building are made of concrete, concrete slabs and asphalt. The floor covering at machine mounting sites most frequently are made of mosaic cement tile, from ceramic tile, and so on.

Quite frequently the floors are washed and the surfaces of the floor in this instance are inclined towards gutters in accordance

with the flooring material. With flooring made of ceramic slabs, concrete, and asphalt, one usually inclines it equal to 2% grade, one inclines an earthen floor to 2-3%.

Problems about the solution of the overhead crane runways, repair and hard stands, light openings, light and ventilation skylights, nonskylight structures and buildings, flat and other roofs, doors, stairs, as far as utilization in mine buildings and construction, do not differ in principle from the solution of conventional industrial buildings, and are not considered here. The problems of hermetically sealing the construction are elucidated in the description of pile drivers and of deckhouse buildings.

Protection of the elements of construction. Under mine conditions one frequently faces difficulties, caused by ground water.

Moisture of a commercial origin and, partly from natural daily operations enters the sites with the ore, during the washing of the equipment, during the washing down of walls and floors, during the operation of blowers and this leads to humidification, the worsening of the thermo-technical indexes of the wall and other barriers, corrosion, warping, rotting, thawing and failure of the designs with a simultaneous spalling and failure of the plastering, change in the color of the staining and so on.

For the protection of the designs against moisture one takes measures to remove the excessive humidity, by the natural and artificial drying of the buildings and construction; one uses hydrophobic substances, electroosmosis, drains, waterproofing and vapor insulation. In this chapter only the problems of insulation are considered.

Techniques for waterproofing the edges of foundations are widely known. Under conditions of the pressures of blasting a cement mortar of a 1:2 composition is usually used as waterproofing for the edges of the foundations and in individual critical cases - a cement mortar with special additives. For small mine construction and

buildings beyond the limits of the two radii seimic dangerous zone, waterproofing can be done by conventional means.

With basements and pits in the ground at the sites, and also in underground industrial protective frames of the designs against moisture, there is usually a provision along the external planes of the foundations. In necessary cases a reliable waterproofing, in full or in part, separates the sunken part of the frame (Figs. 43 and 58). The selection of a means for waterproofing depends primarily upon the degree of moisture in the soils, the level of ground water, the purpose of the construction, and so on. With the washing of floors using appropriate sumps and pumping devices in underground construction the possibility of insignificant filtration of moisture through the insulated surfaces with formation the damp spots, in this case, is permissible. Under these conditions and with crack-resistant designs made using dense concretes, the following waterproofing measures of underground mine construction is possible:

1) waterproofing with the utilization of cold asphalt pastes with a possible hydrostatic pressure of the ground water to 0.5 m in this case, and using three-ply insulation, up to 1.5 m;

2) coated waterproofing with hot bituminous pastes with a possible hydrostatic pressure up to 0.5 m, and with protection and strengthening of insulation using water-proof fabric, up to 2-3 m;

3) asphalt waterproofing, strengthened with fabric along with a possible hydrostatic pressure up to 3 m in this case;

4) waterproofing with a cement-sand mortar along with a possible hydrostatic pressure in using additions of ceresit up to 1 m, and with additions of ceresit to 1 m, and with additions of sodium aluminate to 3 m;

5) waterproofing with a cement-sand mortar using watertight expanding cement along with a possible hydrostatic pressure up to 3 m;

6) waterproofing with a cement-sand mortar with additions of surface-active substances along with a possible hydrostatic pressure up to 5 m;

7) waterproofing using concrete with additions of surface-active substances with a possible hydrostatic pressure up to 7 m in this case;

8) adhesive-like waterproofing with a possible hydrostatic pressure to 5-20 m;

9) waterproofing using vinyl plastics;

10) waterproofing using sheet metal along with a possible hydrostatic pressure up to 50 m.

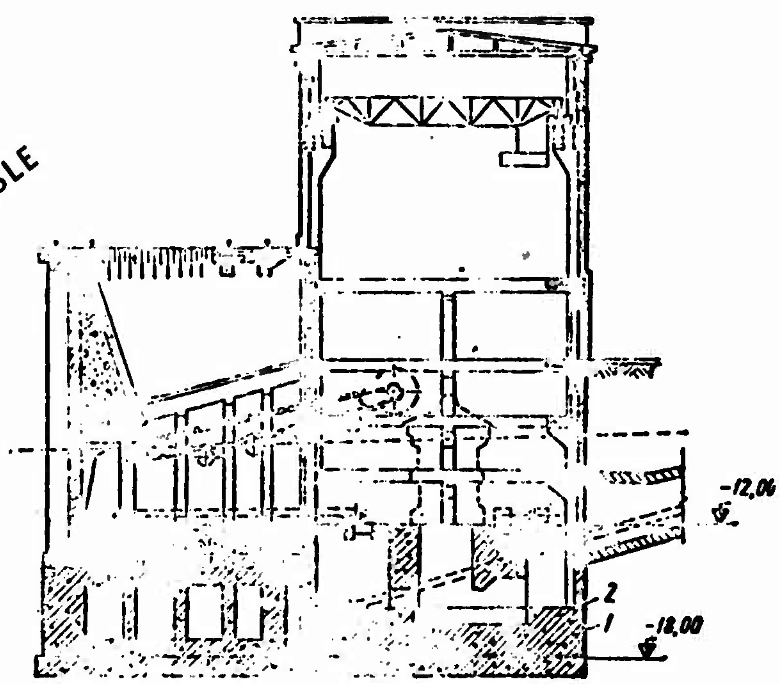

Fig. 58. Partial protection of designs of the sunken part of a housing frame using waterproofing: 1 - ferroconcrete slabs; 2 - waterproofing.

The means of waterproofing enumerated here are briefly described below.

Cold asphalt pastes are applied under conditions of protecting them by insulating them from the pressure surfaces of hydrotechnical structures, and also from the surface insulation of nonpressure head construction. The base of the paste is a bituminous emulsion paste (bitumen BN III - 50%, slaked lime of the first grade - 12%, water - 38%). Cold asphalt pase is a liquid mixture of paste (80%), filler (lime powder with cement to about 17%) and water (3%); following the drying out of the paste waterproof cover is formed.

Cold asphalt paste can be handled as a purified cut and grounded diluted (1:1) paste and as dried concrete and ferroconcrete surfaces. The paste can be applied with a mortar pump at a pressure of not less than 4 [atm(gage)] in a layer from 10 mm (2 threads) to 20 mm (3 threads). By applying the paste by hand the thickness of the thread constitutes 7-10 mm. The protective facing of insulation made of ferroconcrete slabs or cement mortar, applied by means of guniting on a metallic mesh, should have an anchor fastening.

The utilization of the protective cold asphalt paste for the waterproofing of mine construction can be done, especially, in the absence of water pressure and in special cases, with the possibility of the appearance of ground water along with a hydrostatic pressure up to 0.5 m, and with a three-ply insulation up to 1.5 m under conditions of using dense concrete in a direct outlet of the water.

Coated waterproofing using hot bituminous pastes or bitumens can be applied in 2 layers at 2 mm on a smoothed surface, two first coats (cold priming) to the surface from the side of the pressure. This waterproofing can be applied at a hydrostatic pressure of 0.5 m, and with a continuous external protective plastering on the metallic mesh and under a condition of intensifying the insulation glass or asbestos to the fabric, resistant to rotting, the insulation can be used at a pressure up to 2-3 m.

Asphalt waterproofing on horizontal planes can be performed predominantly by applying the concrete preparation to a layer of a

vibrated sandy asphalt concrete, 25-30 mm thick. Asphalt waterproofing of vertical and inclined surfaces, including arched surfaces, is done by applying it in 2 layer, 5-10 mm thick using asphaltomets* or guniting. Waterproofing is done predominantly from the pressure side, and in this case the vertical asphalt waterproofing is protected by an outer protective wall made of a masonry or concrete blocks.

At the joining of the vertical and horizontal asphalt waterproofing one should provide inclined sections and apply waterproof fabrics (fiberglass, asbotkan'** or other fabrics resistant to rotting). During the manufacture of asphalt waterproofing special attention is given to the selection of the composition of asphalt concrete mixtures, its warming up to 200°, the rapidity of applying the mixture on an insulated surface, the utilization of vibrators with a warmed slab, and others. Under the condition of applying the asphalt on the side of the pressure, bolstering the insulation to the fabric and its protection by a preparation, and also stone or concrete walls, asphalt waterproofing can be used with a hydrostatic pressure of the ground water up to 2-3 m.

Waterproofing of cement-sand mortar by using portland cement with additions of ceresit or of sodium aluminate is predominantly done by guniting from the side of the pressure. With the impracticality of guniting, the plastering can be applied in a conventional way. At the junctions of vertical and horizontal planes one ought to provide for inclined sections and insert two metallic meshes made of wire, 1.5-3 mm in diameter with cells, 40 × 40 mm, but with conventional plastering, using cells 10 × 10 mm. The thickness of the plastering amounts to 25 mm, and in sections with metallic meshes - 40 mm. Before applying the plastering, the surface of the design is cut or treated with sand, washed with water and moistened.

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*[Translator's note: this term cannot be found in available sources].

**[Translator's note: this term cannot be found in available sources].

Plastering with mortar having a composition of 1:2 along with a water-cement ratio of 0.4 and less, layers of 8-10 mm are produced. The upper layer with a thickness up to 5 mm is made using fine sand and it is cubbed with cement. On horizontal planes a layer of stiff mortar is applied, which is condensed with the use of vibration. During the course of 7-10 days, and with the additions of sodium aluminate during the course of three days, the surface of the insulation is moistened.

When applying waterproofing using portland cement mortar from the side of the pressure to pressure on a crack-resistant design, the hydrostatic pressure of the ground water with additions of ceresit amount to 1 m, and with additions of sodium aluminate, up to 3 m. The preparation of portland cement mortar in the latter case is characterized by the utilization of water of a 2-3% solution of sodium aluminate. The selection of an accurate concentration of a mortar is done experimentally. The grade of portland cement is 400 and above. The additions of sodium aluminate to plasticized, hydrophobic, pozzolanic portland cement, slag portland cement are not recommended, A mortar with an addition of the sodium aluminate is characterized by rapid gripping, an increase in strength at an early period with a retarded increase in strength subsequently.

Waterproofing of cement-sand mortar using watertight expanding cement is predominantly done by guniting. One applies the waterproofing predominantly to the side of the pressure with the elimination of the filtration of water through the insulated design during the period of productive work. The insulated surface is prepared and washed. Guniting is done with the help of a cement gun. The prepared surface is moistened for three days. The thickness of the plastering, the technique of horizontal insulation are analogous to that described above for waterproofing using portland-cement mortar.

During the guniting of crack-resistant designs, waterproofing with a cement-sand mortar using watertight expanding cement can be carried out at a hydrostatic pressure of the ground water up to 3 m.

Waterproofing with cement-sand mortar along with additions of surface-active substances is characterized by additions of sodium abietate to the extent of 0.02-0.05% together with sulfite-cellulose liquor to the extent of 0.15% of the weight of the cement, considering as a dry substance. For plastering one uses a cement-sand mortar of a 1:2.5-1:3 composition by weight with a water-cement ratio of 0.45 using portland cement or slag portland cement of grade 300. The application of a plaster layer is done using mortar pump, cement gun, and with small volumes - by hand. A mortar to a sufficient degree, is mobile in view of the presence of surface-active additives. With the backwater of the ground water by more than 1-1.5 m, it is recommended to apply interior plastering at a thickness of not less than 5 cm, to fill it in the planking with a vibration, to reinforce the mesh and to fasten with anchors. Interior plastering with additions of surface-active substances was successfully applied for insulating large reservoirs at a hydrostatic pressure of 5 m.

Waterproofing using concrete of increased waterproofness is characterized by additions of surface-active substance with the introduction into the composition of the concrete of sodium abietate to the extent of 0.02% and sulfite-cellulose liquor to the extent of 0.15% calculated based on the dry substance from the weight of cement. The utilization of concrete with additions of surface-active substances is possible at a hydrostatic pressure up to 7 m.

Adhesive waterproofing is prepared from rotproof roofing material, glued on using them along with waterproofing pastes. For adhesive waterproofing hydroisol (predominantly asbestos or asbestos-cellulose cardboard, impregnated and coated with bitumens) and a ruberoid with a sanitized, rotproof base, stuck on using hot bituminous waterproofing pastes made from waterproofing and the other petroleum bitumens are used; also used are uncovered roofing paper (unsanded tar paper) or lightly sanded roofing paper, the sand being stuck on using tarry, waterproofing pastes derived from coal pitch or alloys of pitch with tar, resin and so on.

In critical cases one uses hydroisol; fabrics made from inorganic material, glass cloth and the asbestos fabrics, impregnated with bitumen and coated with refractory bitumen; organic rotresistant fabrics, impregnated and coated with bitumens; rolled sheets made from a mixture of bitumens with asbestos (borulin)*, reinforced in necessary cases with metallic mesh, and so on.

Adhesive waterproofing is usually designed based on a preliminarily smoothened, purified, dry surface and in a number of cases by grounded cold methods with the number of layers of roofing material ranging from two to five, and with the thickness of each layer of hot paste, up to 3 mm.

In the unions of the insulated surfaces, the insulating mat is strengthened by an inorganic fabric having a width up to 1 m, by a metallic mesh or metallic sheets having a thickness up to 1 mm with a width of 0.5 m. The roofing material prior to gluing is held in a rolled out form, and in the presence of a sand sprinkling, it is processed with solvents. The insulating operations are done from the bottom upward; the junctions of the width overlap by 100-150 mm; the mat is fastened to the sanitized strips, in turn, fastened to the insulated design (or to the protected wall). The waterproofing is shielded from the outer side by walls made from clinker masonry, concrete blocks and slabs and is covered with clay joints having a thickness of 0.2-0.3 m. The spaces between the waterproofing and the protective wall are filled with mortar.

Adhesive two-ply waterproofing can be used with a hydrostatic pressure of the ground water up to 5 m, three-ply insulation - with a pressure up to 10 m, and five-ply insulation - at a hydrostatic pressure up to 20 m.

Waterproofing using vinyl plastic, plastic material with a base of polyvinyl chloride resins is designed with protective

*[Translator's note: this term cannot be found in available sources].

wall-screens made of concrete blocks, clinker and others. This type of waterproofing can even be used at small pressures.

The effectiveness of the waterproofing using cement mortar, as well as coated insulation, and the corresponding possible hydrostatic pressure of the ground water due to the failure of using crack-resistant designs, is considerably lowered. For relatively plastic protected asphalt and two- or three-ply adhesive waterproofing, the failure due to the utilization of crack-resistant designs signifies a decrease in the permissible hydrostatic pressure of the ground water by 60-40%; the same, with four or five-ply adhesive insulation corresponds to a decrease in the permissible hydrostatic pressure by 40 and 30%.

Utilization for the purpose of waterproofing dense concrete with an increased waterproofness along with additions of surface-active substances deserves attention because of the simplicity of this method. In mine construction the following basic variants of waterproofing dense concrete are possible:

a) surface-active additives are included in the dense monolithic concrete in the walls of the underground part of the housing (Figs. 43, 58);

b) sectional facing ferroconcrete slabs, made by using dense concrete with surface-active additives are installed on the outer side of the walls of the underground part of the housing. The facing slabs are braced to the reinforcing cage of the caisson, and are used when concreting its walls (just as when erecting the weirs) as plankings. The seams between the slabs are calked with mortar using watertight expanding cement.

With the protection against moisture along the outer walls of the sites under moist and wet conditions adhesive insulation can be applied with a protective screen, which, for example, is necessary and it is also done under conditions of washing at certain industrial

sites. Wide utilization of such complex and relatively expensive insulation in the barriers of industrial sites is not possible.

In Table 8 the methods of protecting the walls of different design under various humid conditions against moisture are reduced.

Table 8.

Simplest means of protecting the walls against moisture	
wall fillings made of brick masonry	wall fillings made of sectional panels with ferroconcrete and prestressed support slabs (Fig. 55c, d)
Sites with a moist condition	
Masonry laid with a plastic cement mortar with the thorough filling of all seams and voids with the pointing of facade seams	Dressing the joints between the slabs with 1:2 cement mortar
Interior plastering with a dense cement mortar	Providing for an increased protective layer of dense concrete for the steel framework, the location of prestressed steel framework in the center of the slab
Painting the interior surfaces of the walls with a resistant coating	
Masonry using well burned grade 100 brick with plastic cement mortar with complete filling of all seams and voids and by the pointing of the facade seams	Dressing the seams between the slabs with cement 1:2 mortar
Interior plastering using dense cement mortar with increased waterproofness	Location of prestressed steel framework in the center of the slab made from dense concrete, and in certain cases, the utilization of concrete with increased waterproofness along with additives
Continuous thorough painting of the interior wall surfaces with resistant coatings (perchlorvinyl resin, dissolved in organic solvents, resistant enamels and paints and others)	Continuous painting of the interior wall surface with resistant coatings

One ought to dwell on one more example of protection against moisture. In some galleries one can observe high relative humidity, reaching 90-100%. As a result of corrosion the metallic designs of span structure of galleries are rapidly destroyed. Under such conditions one ought to predominantly use protective (as was shown above) ferroconcrete support designs. Beam and trusses especially the metallic ones under these conditions can expediently perform beyond the limits of the barrier of the gallery. Actually, for the gallery it is recommended that in this instance to set up right angled locked ferroconcrete frames, set apart from one another at a distance of 6 m lengthwise to the gallery and normal to is axis (Fig. 59). One braces the ferroconcrete flooring slabs facing and support slabs of the wall panels of a gallery to the frames.

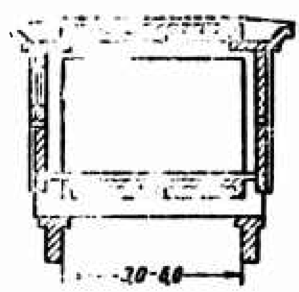

Fig. 59. Diagram of one of the solutions of designs of galleries under conditions of high humidity.

3. Groups of Loads of the Structures and of Their Elements, the Coefficients of the Overload and the Dynamicity

General information about the classification of the loads of construction and their elements and about the data on the values of the coefficients of overload and dynamicity are noted below. The loads of mine construction can be divided into two groups.

The first group pertains to loads based on their own weight of construction and their elements, heat insulation, loose bodies, soil, pay loads of various stockpiles and floorings, snow loads, loads from operating cranes; the additional loads, including wind, loads from repair cranes and temperature, specific action, including seismic loads in the regions of blasting. The remaining loads of the

first group have been repeatedly examined, have been determined by structural standards and by rules, by special and general technical conditions. These loads in subsequent accounts are not clarified in detail and are involved only so far as needed in the examination of special problems.

The loads of the *second group* pertain to special loads of mine construction. Such as, for example, the basic loads of the deckhouse pile drivers and construction of a multirope lift, loads from the weight of the construction, of pulleys, of lift machines and special equipment; the working loads of the lifts, of the load when braking the hauling vessels, at the time of the rupturing of ropes, and so on. The loads of the second group pertain also to special loads from a weight and horizontal pressure of an ore, fluxes and other loose material in hoppers, in ore stockpiles and so on.

A considerable part of the construction and its elements bears dynamic loads during the unloading of the ore in the receiving funnels and at other points, during the movement of railroad cars and rolling stock in deckhouse buildings and haulage construction. A number of structures and their elements undergo the effect of dynamic loads from equipment, set on floorings, related sites and so forth.

The principal loads enumerated here on the construction are examined in later chapters, dedicated to the calculation and construction of mine construction. It should be shown however, that in view of multiplicity and variety of the loads on mine construction, all of the special loads can be covered in the examination of individual structures. It is appropriate, therefore, to present the general data, determining calculated loads on construction and its elements.

In general, the loads on mine construction are set when developing the projects and the working plans of deckhouse buildings and other construction. With the specifications in order to produce the calculations, the determination of stresses, the selection and

check on the sections and joints, it is necessary to know the magnitude of the *coefficients of overload* and of *coefficients of dynamicity.* These values are not constants, but reflect the contemporary practice of producing calculations. Given in Table 9 are the values of the most commonly used coefficients of overload and coefficients of dynamicity, taken when making the calculations of mine pile drivers, construction, deckhouse buildings, hauling galleries and piers, conveyor galleries, hoppers and other construction of the mining industry. It is necessary to show that in certain cases (and subsequently - everything in the greater number of cases) the dynamic action of the loads should be determined only during the process of the dynamic calculation of the construction. Thus, for instance, the dynamic action of the loads from equipment set on the flooring of a multirope lift is determined. For such cases the values of the dynamic coefficients are not given in Table 9.

Table 9.

Load and construction	Coefficients		
	overload $K_{п}$	dynamicity $K_{д}$	$K = K_{п} \cdot K_{д}$
A. Overall loads			
Intrinsic weight of the construction, apart from the weight of the heat insulation........................	1.1	-	1.1
Weight of the heat insulation.....	1.2	-	1.2
Effect of the snow................	1.4	-	1.4
Effect of the wind................	1.2	-	1.2
Vertical and horizontal loads from the cranes........................	1.3	1.2	1.6
Loads from the telphers...........	1.3	1.1	1.4
Loads from lift carriages.........	1.3	-	1.3
Loads at industrial sites and stockpiles.........................	1.2	-	1.2
Loads on serving grounds..........	1.3	-	1.3
Hydrostatic pressure of the liquids............................	1.1	-	1.1
Effect of temperature.............	1.2	-	1.2

Table 9 (Cont'd.).

Load and construction	Coefficients		
	overload $K_{П}$	dynamicity $K_{Д}$	$K = K_{П} \cdot K_{Д}$
Seismic loads of high structures in the zones of the effect of blasting:			
at the top of the structure.....	-	2.0	2.0
at the edge of the foundations..	-	1.0	1.0
B. Loads of deckhouse pile drivers			
Working loads of the lift cage in calculation of main trusses and frames..............................	1.3	1.1	1.4
Working loads of the skip lift in the calculation of main trusses and frames..............................	1.3	-	1.3
Working loads of a lift cage in the calculation of bearing and transverse trusses and beams........	1.3	1.25	1.6
Working loads of the skip lift in the calculation of bearing and transverse trusses and beams:			
at an acceleration up to 1.0 m/s^2....................	1.2	1.3	1.6
at an acceleration up to 1.25 m/s^2...................	1.2	1.35	1.6
Load when the rope ruptures.......	1.4	-	1.4
Intrinsic weight of the designs and the weight of the ropes.........	1.1	-	1.1
Weight of the new and insufficiently-studied designs......	1.2	-	1.2
Effect of the landing of a case on the dogs with the additional combination of loads not taking into account the rigidity of the cage....	1.2	4	5.0
The same, using the basic combination of the loads..............	1.2	2.5	3.0
Effect of the landing of the cage on the dogs allowing for the ridigity of the case and underdog beams (in the calculation of the cage-beam system)..................	1.3	According to calculation	

Table 9 (Cont'd.).

Load and construction	Coefficients		
	overload $K_{П}$	dynamicity $K_{Д}$	$K = K_{П} \cdot K_{Д}$
Effect of the braking cage with its fall following rupture of the rope..............................	1.5	According to specifications and datum, presented in Chapter V	
Effect of the loads of the sinking equipment......................	Averaged value, 1.3	-	-

C. Loads of the deckhouse buildings and of hauling galleries

Pressure in the receiving skip funnels during the loading of small-size fregments of crushed ore...............................	1.3	1.2-1.15	1.5
Loads from rolling stock: for metallic span structures along with the calculation of a prospective increase of the loads by 10-20%................	1.3-1.4	Coefficients of dynamicity are assumed depending on the ballast, length of the load and the rest (see Chapters VIII and IX)	
for ferroconcrete span structures with the calculation of a prospective increase of loads by 20-30%....................	1.4-1.5		
Evenly distributed temporary loads of the flooring and of sites of the deckhouse building..	1.3	-	1.3
Weight of the ballast-heat insulation.............................	1.2	-	1.2
Intrinsic weight of the designs..	1.1	-	1.1
The same, with the calculation of the weight of small pipes manifolds and conduits.....................	1.2	-	1.2

Table 9 (Cont'd.).

Load and construction	Coefficients		
	overload $K_П$	dynamicity $K_Д$	$K = K_П \cdot K_Д$
D. The loads of hoppers and funnels			
Pressure of loose material in the receiving hoppers and funnels during the unloading of lumpy ore from the dump cars and dump trucks with a carrying capacity of 25 t or more with a depth of the funnel of more than 8 m......................	1.25	2.0	2.5
The same, with the ore of average lumpiness......................	1.3	1.5	2.0
Pressure of the loose material in the receiving hoppers and funnels during the unloading of small-size fragments of crushed ores from trams and dump trucks having a carrying capacity of 10 t or less with the depth of the funnel at 8 m and less......................	1.3	1.2-1.5	1.5
Pressure of the loose material in the hoppers and funnels during loading of conveyors and in the absence of precise data about volumetric weight.................	1.3	-	1.3
The same, with precise data on the value of the greatest volumetric weight of the ore and other loose material...................	1.2	-	1.2
E. Loads of the galleries and of sites of conveyors and feeders			
Loads from belt and shuttle conveyors:			
an average segment, terminal stations, mobile unloading carts of belt and shuttle conveyors.......................	1.3	1.1-1.2	1.5-1.6
driving stations..............	1.3	1.5	2.0
tension devices and longitudinal tension of the belt conveyors.......................	1.2	-	1.2

Table 9 (Cont'd.).

Load and construction	Coefficients		
	overload $K_{п}$	dynamicity $K_{д}$	$K = K_{п} \cdot K_{д}$
Temporary evenly distributed load of the conveyor galleries..........	1.2	-	1.2
Weight of platform feeders and their drives......................	1.3	1.5	2.0
Temporary evenly distributed load of the site.......................	1.3	-	1.3

4. Certain Problems in the Building of Mine Construction

During the building of ore open pits, shafts, and ore-concentration combines of enterprises the following general problems should be solved by interrelationships: the building of railroad lines, roads and the construction of transport; the laying of electro-transmission, couplings, district heating, compressed air, water pipe, sludge, sewerage and other conduits along with their linear construction; the realizing the objectives of the structural base, temporary buildings, and others; organization of the sinking or stripping operation; the erection of construction and buildings at the sites of pits and at the workers' settlement.

Most complex is the linking up of these problems during the building of the buildings and construction at the industrial site shafts. In this instance the thorough studying and co-ordination of the calendar dates of the erection of structures, providing the most expedient mutual location of the construction for power driving and other temporary and permanent construction of an open pit, is especially necessary. Given above were the comparatively more (Figs. 15 and 22) or less (Figs. 14 and 20) successful diagrams of the location of the structure of ore mine shafts. Diagrams in Figs. 15 and 22 allow to the greatest degree for a combination of work through the driving shafts and the erection of permanent buildings and structures at the surface of the ore mine shaft.

In each individual case during the resolution of the problems of construction technology at the site of the mine shaft one ought to consider along with volumes and sequence, the procedures of mining and construction-maintenance work, at the level of their mechanization, the character of the applied designs of the mine construction and the established dates of their erection, and also along with the specific conditions which, in part are stated below.

General construction plans are developed for several characteristic periods, the first of which is the *preparatory* period. The *second* period is characterized by intensive driving, and also by the building of the first priority construction. At two basic units of the industrial buildings and construction (Fig. 15) during this period the construction of the unit of the buildings is carried out. The construction of the second unit is done during the *third* period. Also in a number of cases using a combination of the periods, the appropriate unification of the general construction plan is carried out.

During the preparatory period the earth work is done, the railroad lines and roads are built, the communication lines, electro-transmission lines, temporary or permanent electric substations, temporary construction and stockpiles, including material warehouses and storage houses for the explosives, sinking construction, part of the permanent buildings, utilized for the purpose of driving and the construction, primarily the first order administrative offices of enterprises and mechanical workshops; they install the sinking equipment and construction mechanisms. During this same period in a number of cases the mouth of the shaft is sunk. One solves the last problem depending on the local conditions.

With drum lift machines and pile drivers with guide pulleys the driving of the mouth of the shaft with the erection of shoring is done prior to installing sinking pile driver using temporary suspension shoring excavator or crane with a bucket, mobile hopper and dump trucks. When using temporary shoring in weak rock,

the mouth of the shaft which is determined by the limit of the stability of conventional temporary shoring and by difficulties with the equipment for the supporting rim usually does not extend more than 8-10 m. Under such conditions the rapid erection of permanent shoring is necessary. When tubing the shoring, for example, it is recommended that the soil be burrowed to a depth of 3-5 m, and this section be fastened with the concrete to the remainder of the recesses for the pile driver underframe, and the cementing of the anchors for the suspension of the tubes.

With multirope lift devices the shoring of the mouth of the shaft can be combined with the foundation of the structure. In this instance, reliable tubing or other shoring, under examination during the erection of the permanent pile driver should be provided in the section of the foundation of a tower pile driver in the sha t. The shoring of this section is suspended to the temporary supporting concrete rim, located at the level of the surface or lower, or to the beam flooring of the cage, which lies on shallow foundation slabs. At the level of the foundation of the subsequently discussed section of shoring it is expedient to set up a developed ferroconcrete ring by design. The presence of the latter results in the elimination of the need to reinforce the section of permanent shoring.

During the building of mine shafts the problems dealing with the most expedient driving of the shaft for horizontal mining are solved. In accordance with this, permanent pile drivers with guide pulleys, in certain cases, are used for purposes of driving.

The sizes of permanent pile driver machines by design are relatively small, as a consequence of which its positioning at the level of the head within the limits of the permanent pile driver machine of a large number of pulleys, necessary for the suspension of the sinking equipment, usually is not possible. At the subpulley site of the permanent pile driver during the driving subpulley beams and pulleys are installed for lifting the buckets and suspension pumps. The pulleys for the suspension of rescue stairs, cables and

some other items are installed on a temporary intermediate landing. Conduits for cementation, ventilation and compressed air are suspended to the buntons of the shaft. The sinking landing is suspended in this instance by two branches of a rope with the bracing of one branch to the subpile driving frame and by passing the second branch through the pulley, set on a special landing in the mouth of the shaft. Thus, the utilization of the permanent pile drivers for the purposes of driving the shaft means the need for utilizing the specific technology of driving, which should be thoroughly studied.

In general, the utilization of permanent pile drivers for the purpose of drivings is relatively expedient with a minimum number of pulleys, i.e., for small shafts. Under conditions of driving deep shafts the utilization of permanent pile drivers results in the limitation of high-speed indexes of driving.

In most cases the driving is done using stock add-on sectional sinking pile drivers. In this case usually the problem of the rapid replacement of the sinking pile driver by a permanent one should be solved. The metallic permanent deckhouse pile drivers answer this condition to the highest degree. During the replacement of the sinking pile driver by a permanent one a complex of problems should be solved, which arises, beginning with the moment of stopping the operation in driving the mine shaft and of the crosscuts of the near-shaft yards up to the moment of adjustment of the permanent lift. Except for the time to complete the operations on positioning or installation the permanent pile driver, one ought to consider the time to hoist the sinking equipment and dismantling the landing area, ropes, pulleys and a sinking pile driver, for which 5-10 days is necessary. Furthermore, the subpile driver frame, alignment of the pile driver, coupling the machine to the subpile driver frame, the installation of dogs and the attachment of the vessels an additional 10-12 days is necessary.

For the thoroughly organized positioning of the pile driver a considerable period is not necessary. The positioning of one of

the mine metallic pile drivers at a height of about 50 m and at a weight of more than 100 t was performed after 5 h at an average rate of the movement of a pile driver at about 5 m/h.

Several methods of installing the metallic deckhouse pile drivers with guide pulleys are known. One of the simplest methods amounts to installing an assembled pile driver in a horizontal position with its movement to the shaft and by the further lifting and turning of the machine, and boom of the pile driver around on three hinges into the assigned position. The widely-known method of installing the assembled machine in the horizontal position with its movement and lift over the shaft with the simultaneous sliding of the lower part of the machine along guides, the mounting of the machine on the subpile driver beam with the further mounting of the boom. Under the condition of a preliminarily made expanded assembling of the designs, the installation of the average pile driver using the specified methods can be made over 2-6 days.

With the calculation of the above enumerated work for replacing the sinking lift with a permanent one, installing the pile driver with guide pulleys and adjusting the lift under ordinary conditions, 1-1.5 months is necessary.

Specifically, the problem of replacing the pile driver with permanent tower pile drivers with multirope lift devices is taken into consideration here. In particular cases, for metallic tower pile drivers and small ferroconcrete pile driver, the possibility of productive work also by a method of positioning the structure has not been excluded. The weight of the supporting metallic designs of the mine tower pile drivers constitutes 300-3000 t; in this case, the weight of 1000 t and more corresponds to heavy tower pile drivers.

Today, there are several examples of the practical realization moving construction at a speed up to 10 m/h with the weight of the moving structures going up to 3500 t. At one of the iron-ore mines of the USSR for the first time in the world the feat of moving a

mounted metallic tower pile driver weighing 3500 t with four supporting multirope lift devices for a distance of 52 m from the mine shaft was accomplished. The move was made over several hours.

In general during the replacement of a sinking pile driver by a permanent tower pile driver measures are carried out, directed towards a maximum reduction of the periods of erecting the permanent pile driver (the utilization of mobile planking and others) and phased with the time of mining work. In this case when heading the horizontal mine workings during the period of the erection of the tower pile driver it should be possible to use the nearest shafts.

During the erection of the ferroconcrete tower pile driver with the multirope lift devices or ferroconcrete pile driver with guide pulleys, a nonpile driving heading of the shaft by parallel completed work can be used in driving and erecting the permanent pile driver. This type of driving was organized in the coal industry for driving vertical shafts having a diameter of 7.5 m. In this case at a depth 22 m below the level of the shaft mouth a landing was located, where the rock from the bucket in the trolleys, is taken to the surface along inclined workings with a double-end inclined lift. On the surface the rock is unloaded into a hopper and then into dump trucks. The mine shaft is floored below the subpile driver frame which makes it possible to simultaneously operate the driving of the shaft and the erection of the driving hopper - ferroconcrete nonboom pile driver with guide pulleys and a receiving hopper.

One ought to note, that the increase in the volume of the workings manufacture using a nonpile driving heading, located under the foundations of the tower pile driver or driving hopper, is deficient, the consequences of which - the possibility of the settling of the construction - should be forewarned by all necessary means.

Sometimes the utilization and tower pile drivers for the purpose of driving the workings is not ruled out, which, by corresponding form, changes the above given positions.

Among the other measures, which successfully allow one to solve the problems of mine construction, one ought to note the following:

1) the reduction of the time periods of erection of the main construction, which determines the time of initiating the operations for the undertaking;

2) the organization of the productive work in the central mine units, including the erection of deckhouse pile drivers and other structures going to a height of 100 m;

3) the possibilities of combining the erection-installation work, which takes on special significance in space-related units;

4) moving of construction and its units weighing up to 5000 t or more in central units of mine shaft and of units of other construction predominantly with limited sizes by design and which are characterized by considerable erection-installation time;

5) the erection of tower ferroconcrete pile drivers and of other high structures using mobile planking;

6) the utilization of steel framework for bracing the shields of the planking, ferroconcrete and light-weight concrete sheathing units.

In planning mine construction also these problems should be thoroughly developed:

1) the mechanization of the supply and laying of concrete with its descent to a considerable depth in the forms of receiving funnels, crushing housing, the shoring of the mouth and stems of mine shafts and by the lifting of concrete to the upper levels of tower pile drivers and other high construction;

2) the installation of the structures of deckhouse buildings

and also hauling and conveyor galleries with spans up to 50-100 m or more;

3) the installation of heavy structures of hoppers and ore storage bins;

4) equipment and installation of contemporary structure of wall and other barriers with the calculation of their protection against the action of an unfavorable temperature-moisture condition, vibrations, explosions, abrasions, etc.;

5) the erection of tunnels and masonry of communication with their linear construcion;

6) installations varied in purpose, based on the weight and the over-all sizes of the equipment, located at various levels in the locality of the construction, in housings, galleries, underground workings and open areas;

7) earthen work in rocky and other soils in trenches of considerable depth;

8) the utilization of caissons and other foundations set deep;

9) the erection of structural foundations as revetments to the stems of mine shafts and other sunken designs;

10) the utilization of assemble-disassemble foundations for the largest part of the sinking equipment, and others.

CHAPTER VI

DESIGNS OF PILE DRIVERS

1. General Information

The accepted schemes of lifts, the system of the pile driver and material from which it is built primarily affect the choice of design of the pile driver. Today metallic pile drivers are predominantly used, and in a number of cases the utilization of other construction material is found. The presence of specified material or the absence of some of them in the construction field, the periods of construction connected with the continuous operation of the undertaking and the specific conditions of the construction of the deckhouse construction, largely determine the selection and practical final outcome of the design of the pile driver.

Presented in this chapter is a description of the designs of metallic, steel-reinforced cement, ferroconcrete and wooden mine pile drivers; attention is given to the description of the elements of the metallic pile drivers, in which the characteristics of the systems are most clearly reflected. Therefore, the description of the elements of metallic pile drivers simultaneously has significance even for the characteristic of the designs and of the elements of the pile drivers made from other materials, primarily steel-reinforced concrete and sectional ferroconcrete. Described below are the elements and the parts of pile drivers, including the designs of machine installations, their facing, parts of the underdog beams,

buntons and conduits, and with some changes the parts are repeated in tower pile drivers, which are used during multirope lifts.

Information on lift vessels and parachutes. The main single rope lifts of ore mines, used for lifting ore and in large mines and for the lifting of rocks are usually equipped with overturning skips along with payloads of 2; 4; 6; 8 and 10 m^3, which corresponds to carrying capacities of 5; 7.5; 10; 15; 20 and 25 t. The skips with bottom unloading are used extensively with multirope lifts and have a carrying capacity up to 50 t.

In order to fulfill auxiliary operations (transport-lifting people, rocks, material and equipment), and in mines of low productivity and for the lifting of ore, usually nonoverturning cages are used. The overturning cages, widely used in the coal industry, are rarely used in ore mines.

The unloading of the overturning skips is done with the help of special unloading curves (Fig. 83).

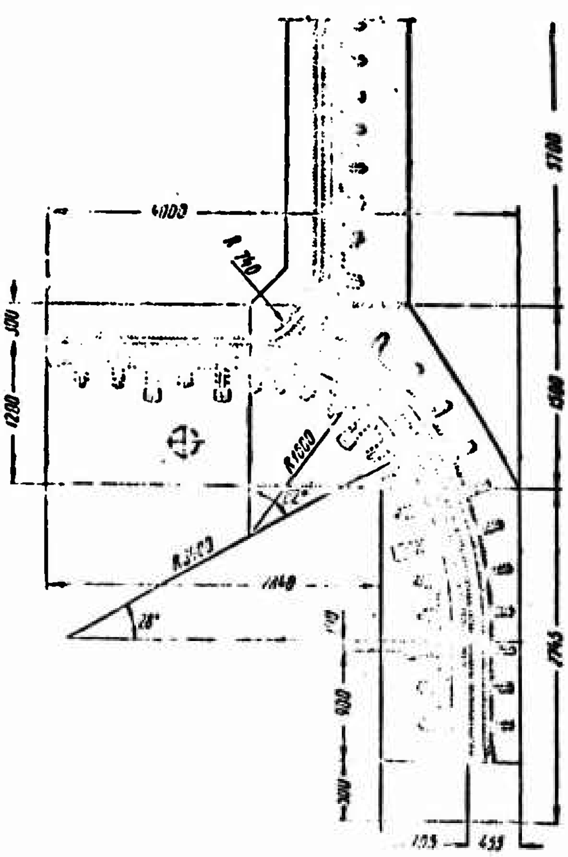

Fig. 83. Curves for unloading the skips.

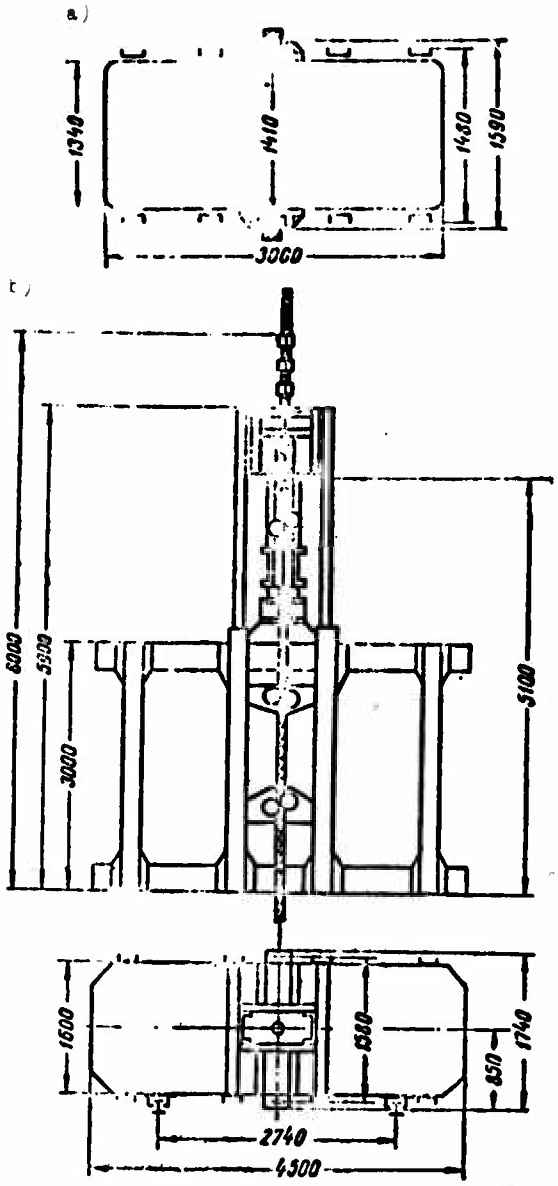

Fig. 84. Mine cages: a) the diagram of a single-deck cage with wooden guides for the ore trolley having a capacity of 1 m^3 (2.5 t); b) the diagram of a cage with metallic guides and PTK parachutes for an ore trolley having a capacity of 4 m^3 (10 t).

During unloading the overturning skips and cages are inclined in a horizontal direction for a considerable distance, in connection with the fact that the unloading curves are attached to a special, contilever part of the pile driver machine, called an unloading cantilever.

In the ore industry single-deck and double deck cages are used. The mine cages have been calculated for lift loaded trams having a capacity of 0.5-10.0 m^3, which corresponds to a carrying capacity of 1.2-25 t. The most widely used standardized capacities of overturning ore trams (VRO) [ВРО] – 0.5; 0.- and 1.0 m^3; self-dumping ore trams (VRS) [ВРС] – 1.6; 2.5; 3.4 m^3; blind ore trams (VRG) [ВРГ] – 1.2; 2; 4; 6; 8; 10 m^3;

The frame of the cage – usually steel, in certain cases light alloys are used. The closet is sheathed with a perforated metallic sheet; inside the cage, as a rule, the railway for the horizontal movement of the trams is located.

Figure 84a, b, shows the diagrams of the widely used, single-deck cages with wooden and metallic guides.

Lifts with conventional heavy cages (with end loads of more than 10-15 t) under mine conditions are supplied by parachutes with brake ropes [PTK] (ПТК). These parachutes operate for any end load and practically at any depth of a shaft.

The diagrams of the location of the brake ropes and pile driver guide pulleys is shown in Fig. 85. With two-sided guides (Fig. 85a, c) the brake ropes are positioned in the plane, passing through the center of the lift rope and make up by design a certain angle with the plane of the guides, but with one-sided conductors (Fig. 85c, d) the brake ropes are located in the plane, passing through the center of the lift rope, parallel to the sides of a machine mount and lifting vessel.

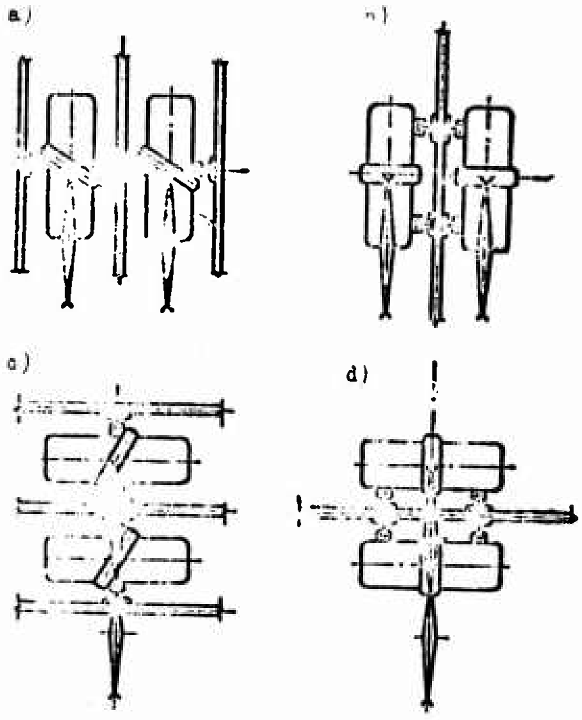

Fig. 85. Basic diagrams of the position of the guide pulleys and brake ropes using PTK parachutes: a), b) with the position of the guide pulleys at one level; c), d) with the position of the guide pulleys along one vertical plane.

The upper ends of the brake ropes end up at the pile driver group of parachute equipment, where it includes shock absorbers, connecting clutches and plates under shock absorbers.

Figure 86 shows the attachment of the brake ropes on a lower subpulley landing with the position of the pulleys in one vertical plane.

In this instance the lower pulley prevents the coupling of the one outer brake rope, in connection with the fact that with two sides of the pulley symmetrical relative to its axes, the two shock absorbers are installed; the outer brake rope is fastened to the shock-proof ropes or to the arm.

The difficulties using the outer shock absorber can be completely eliminated by positioning the shock absorbers on a special landing. The load from the brake ropes on the pile driver is transmitted consequently through the connecting clutches, shock-proof ropes,

shock absorbers and their plates. The introduction of relatively small shock-proof ropes can be explained by their wear and by the wear and tear of the shock absorbers and by the need for replacing them.

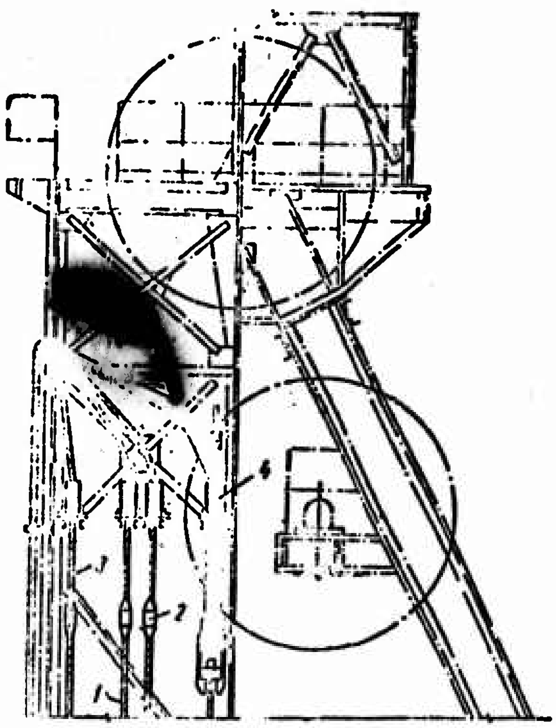

Fig. 86. Diagram of the location of the pile driver group of the PTK braking devices with the location of the pulleys in one vertical plane: 1 – brake ropes; 2 – connecting clutches; 3 – shock-proof ropes; 4 – shock absorbers.

The shock-proof ropes are cramped into the spiral-rope in shock absorber and they have an undulating form. The brake rope is suspended by one, two or three shock-proof ropes; in the latter case, one of the central ropes directly sustains the load of the brake rope, and the two others are free, with slack. With arise of considerable strengthening of braking in the brake ropes the latter is transmitted primarily to one shock-proof rope. With a certain tightening of the central shock-proof rope through the shock absorber, considerable resistance is created because of the friction between undulating form of cramping of the rope and the working surfaces of the shock absorbers. The resistance of the shock absorber can be regulated within rather wide limits because of the movement of the screw gripper. In proportion to the tightening of the first shock-proof rope during the operation the remaining ropes are collected, which

are also cramped in the shock absorber. The described shock absorbers assure the necessary resistances and the conditions of braking the cage. Thus, for instance, if a cage with a man in it is braked during a wreck with deceleration with the limits of 5g, then during the abrupt cut-off of the same cage with some people in it would experience somewhat less deceleration, i.e., conditions would be created more favorable for a human organism. The existing designs of spiral-rope shock absorbers assure a change in the values of the resistance over wide limits. The magnitude of resistance basically depends upon the kind of rope in the shock absorber, i.e., the position of the screw and, to a small degree, how it changes with various conditions of the surfaces of the ropes and wheels of the shock absorber.

The cage groups of the PTK parachute equipment consists of a parachute catcher, suspension devices, guide clutches and the terminals of the lift rope. During the breakaway of the lift rope, the cage group equipment assures the catching of a falling cage and the transmission of the loads from the cage through the catcher to the brake ropes.

In order to create the necessary working tension of the brake ropes as well as to set the positions of their lower ends a sump group of parachute devices, consisting of a tension limiter, a tension clamp, beams and a terminal for the brake rope is installed.

The general concept about the beam cage at the pile driver site, involves shock absorber devices under normal haulage and with the location of the pulleys in one vertical plane, as given in Fig. 87. On this basis two sets of shock absorbers, with two shock absorbers in each set were established. Each shock absorber, in this case, considering the central location of the conductors, shifts of a lift by 384 mm from the axis which determines the position of the beams, supports shock absorbers and sustains the loads of braking devices.

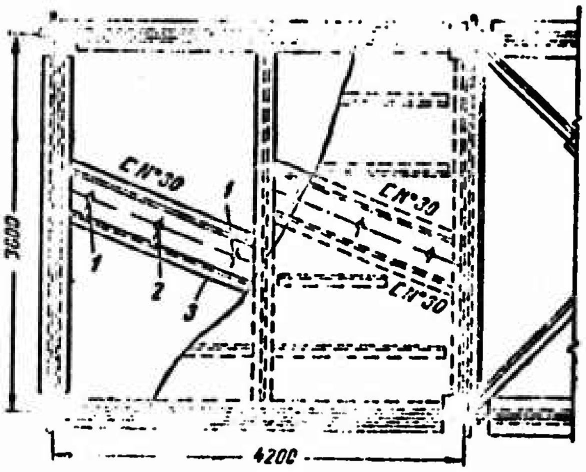

Fig. 87. The shock absorber landing device: 1 – the positioning point of the brake ropes; 2 – lift rope; 3 – beam, which supports the shock absorbers.

Usually, single-groove and triple-groove twin-screw shock absorbers with shock adsorber ropes having a diameter of 43.5 mm, are used. Using triple-groove shock absorbers three or two shock adsorber ropes with brake ropes having a diameter of 31-52 mm are installed. With brake ropes having a diameter of 40 mm and more, three shock adsorber ropes are installed. Triple-groove twin-screw TASM2-1 shock absorber has the sizes of the supporting plate, 340 × 430 mm, with the grid of the anchor bolts at 290 × 270 mm. The shock adsorber ropes are displaced from the center of the larger side of the support plate by 15 mm. Single-groove twin-screw TASM1-1 shock absorber differs from the triple-groove by less than the width of a support plate (252 mm instead of 340 mm) and by less distance between the axes of anchor bolts (202 mm instead of 290 mm), respectively. The height of the twin-screw shock absorbers is 920 mm. The weight of the single-groove shock absorber is 0.2 t, the triple-groove, is 0.3 t. The shock absorbers are usually installed on the transverse channel bars, whose walls are inverted toward the axis of the rope, the distance between the graduation line of the shelf of channel bars for these shock absorbers constitutes 270 mm.

For cage lifts with one-sided rail conductors [PKL] (ПКЛ) parachutes (the parachutes of the cage are lightweight) are used. By installing them inorder to catch the cage, a one brake rope is used, which is located between the rail conductors on one side of the cage. The brake rope is gripped in the rope-screw PTK shock

absorber, set on the lower pulley or on a special landing of the pile driver. The parts of the equipment and of the shock absorber device in this instance completely correspond to that described above.

With rail conductors and end loads up to 15 t block-eccentric [RKE] (РКЭ) rail or rail eccentric-shoe [REK] (РЭК) parachutes are used, which are characterized by positioning the rope-screw of the shock absorbers to the cage; there is not need for brake ropes and the shock absorbers on the pile driver landing in this instance.

With wooden conductors and end loads up to 15 t, parachutes made for the wooden conductors [PDP] (ПДП) are used which, have rope-screw shock absorbers on the cage, and do not require the suspension of the brake ropes and installation of the shock absorbers on the pile driver landing.

Wooden conductors has not precluded the use of parachutes for catching (Shakhtostroy's parachutes designed for wooden conductors). During a breakaway of the lift rope the crabs (the seizures of the parachutes) are made by a main spring by a motion which is turned at a certain angle and cuts into the wooden conductors. In this way the braking of the falling cage and its stoppage is attained. A number of cages for lifting ore trams having a capacity up to 1 m^3 has been equipped with notching parachutes.

In view of the nonuniformity of the sturcture of the wood for other reasons as well, notching parachutes are characterized by the possibility of the emergence of an inadmissible amount of braking forces and the deceleration of the cage. Therefore, the parachutes of this type can be used for small end loads (up to 10 t) and with a ratio of the weight of the loaded cage to its dead weight, up to 1.8.

Pulleys and pulley landings. On the pulley landing guide pulleys are installed; at the level of the pulley landing or near it, shock absorbers of PTK, PKL parachutes and other equipment of the pile driver group of parachutes are placed. Routine inspections of the lubrication of the pulleys and parachute equipment are made at the pulley landing. Near the level of the pulley landing in a number of cases stop bars and other devices, connected with the warning of an accident during an overwind are installed.

The pile drivers of small and average size are equipped with pulleys 1200-4000 mm in diameter. On the larger pile drivers pulleys 4.0-6.0 mm in diameter are commonly installed.

Table 22 gives some information about guide pulleys and the diameters corresponding to them and the rupturing forces of the lift ropes.

Table 22.

Diameter of the pulley D, mm	When $\frac{D}{d} > 100$			When $\frac{D}{d} > 80$		
	diameter of the rope d, mm	Rupturing force of the strands of rope at ultimate strength		diameter of the rope d, mm	Rupture force of the strands of rope at ultimate strength	
		160 kg/mm²	180 kg/mm²		160 kg/mm²	180 kg/mm²
1200	11	7 020	7 900	14	11 600	13 050
1600	15,5	14 300	16 100	20	24 200	27 200
2000	20	24 200	27 200	25	35 650	41 200
2500	25	36 650	41 200	31	57 250	64 400
	26,5	41 400	46 550			
3000	31	57 250	64 400	37	82 400	92 750
4000	37	82 400	92 750	46,5	128 500	145 000
4000	39	90 450	101 500	47,5	135 000	152 000
5000	47,5	135 000	152 000	60,5	218 500	246 000
6000	56,5	180 000	212 500	65	251 500	—

The loads from the pulleys are transmitted through the support and end planes of the bearings of the underpulley truss. The bearing is usually installed on an upper belt of the underpulley truss in the joint of the latter taking into account that resultant forces

in the ropes, transmitted by the bearing, would pass through the center of the pulley and the center of the joint of the underpulley truss (Fig. 88). Direct support of the bearing is provided by the support plate made from sheet metal having a thickness of 12-40 mm (usually 20 mm). The plate is attached to the upper strap of the truss and joined to its elements by riveting or welding. The bearing is braced to the plate with bolts and wedged between two bearing sheets having a thickness of 30; 40 or 50 mm. The bearing sheets are commonly made of components from two strips, each having a thickness of 20 mm.

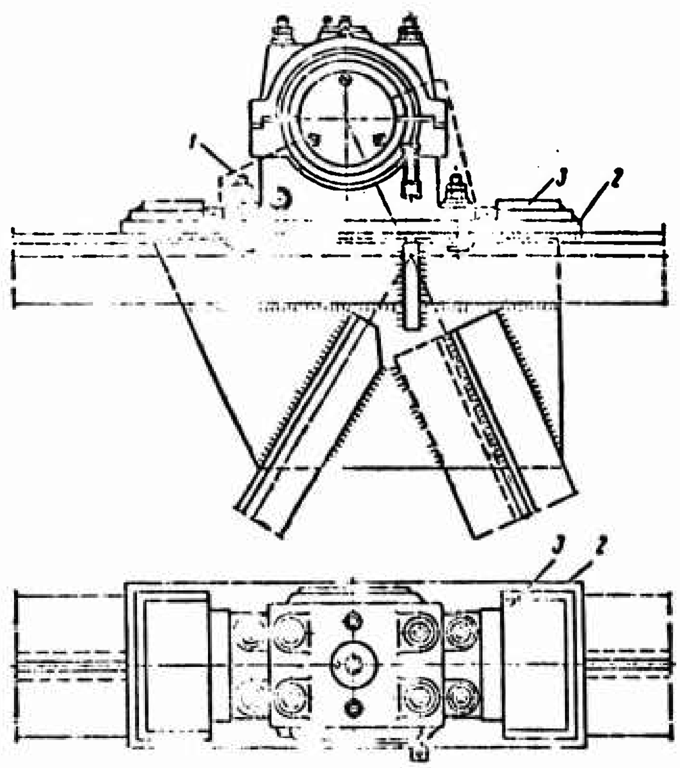

Fig. 88. Diagram of the bearing device guiding the pulley: 1 – bearing; 2 – support plate; 3 – support sheets.

The bearings are installed symmetrically relative to the underpulley trusses in a transverse direction. Thus, the vertical plane of symmetry of the truss is usually parallel to the planes of the pulley and passes through the center of the bearing. Sometimes, the plane of the pulley constitutes a small angle in the plane with the vertical planes of the trusses. However, even in this case, the center of the bearing rests in the plane of the truss or in the immediate vicinity of the latter.

The weight of the complete set of small pulleys, installed at the underpulley landing, having a diameter of 1.2-2.0 m, including the weight of the bearings and axes of the pulleys, constitutes 0.5-1.4 t. The corresponding weight of the average pulleys having a diameter of 2.5-4.0 m constitutes 1.5-5 t, and with diameters of 5-6 m, attains 6.7-12.0 t.

For lift devices and the repair of pulleys a simple assembly frame (Fig. 89) is usually used, the uprights of which are located in the plane of main trusses or frames of the pile driver with the distance between their arms corresponding to the amount of panel below the positioned design. The crosspiece of a transverse frame is made of two parallel aligned beams, joined by horizontal couplings. The beams for the installation of the pulleys are set parallel to the planes of the main pile driver frame, for the most part, in the planes of the pulleys. If the number of pulleys and their weight are considerable, and the underpulley landing is developed according to plan, then it is possible to use a bridge crane for the installation of the pulleys.

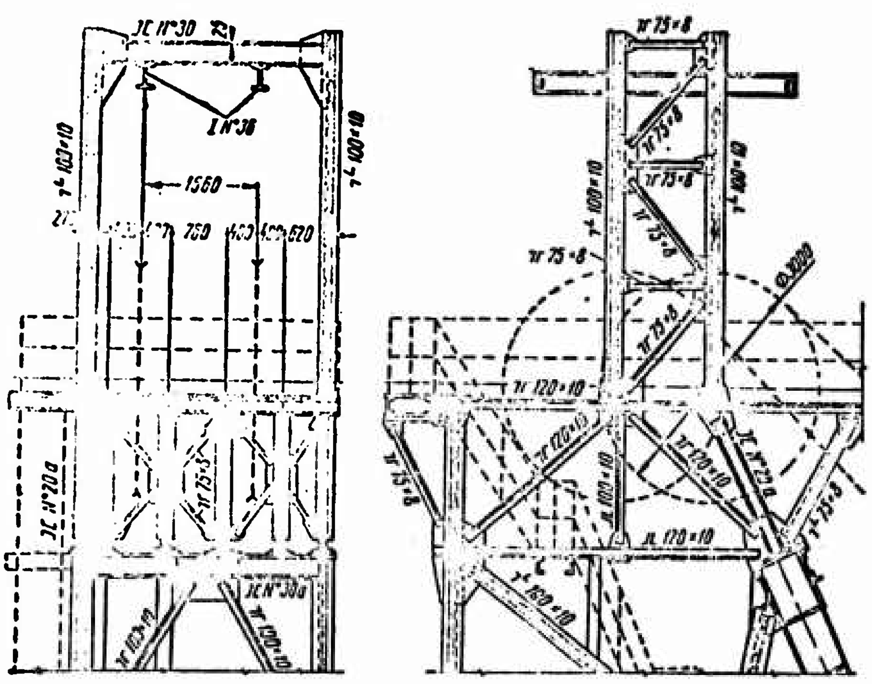

Fig. 89, Assembly frame for the lift, installation and repair of guide pulleys.

Devices for overwinding. During the lift and descent of people to a depth of 400 m and more in a shaft the cage moves at a speed of up to 12 m/s. The rate of movement of the skips at the height of the lift to 1 km can reach 25 m/s (90 km/h). Such velocities are necessary to meet the assigned productivity of the undertaking. The safety of the operation is secured, especially, by the utilization of devices, which prevent the possibility of overwinding. For this purpose the lift device is equipped with two end switches, set on a pile driver and on the indicator of the depth of the lift machine, automatically cutting off the device which includes a safety brake during the lifting of the cage by 0.5 m higher than the normal position when unloading.

With the installation of automatic safety devices, the height of overwinding for cage lift devices should not be less than 4 and 6 m at rates of liftup to 3 m/s and more than 3 m/s, respectively. For cargo lifts using skips, the height of overwinding should be set for not less than 2.5 m.

The contact end switches installed on the pile driver are engaged by direct pressure of the lift vessel on a special lever during the overwinding of a cage or skip by 0.5 m. In a corresponding way the end switches on the indicator of the depth in a machine building are also adjusted. The latter are quite accessible for inspection, do not experience shock, are located indoors at a constant positive temperature, operate reliably; under operation; conditions of the contact of the end switches on the pile driver are unfavorable: their inspection is hampered, and under conditions of high humidity at a low temperature icing occurs. These drawbacks do not have noncontact end switches, which have been recently made and extensively used. Just as with other end switches, they are completely compact and they do not require much space in a pile driver.

One should point out that prior to the extensive use of end switches designed for overwinding various devices were employed. The possibility of overwinding was prevented by means of:

1) the convergence of the strands (during overwinding a wedging of the lift vessel takes place due to the strands);

2) the convergence of the strands with their strengthening by special designs;

3) the installation of emergency catch-latches;

4) the installation of stop (support) rod;

5) utilization of harpoons and so on.

According to the rules of safety it is necessary to use some of the shown devices under the condition that the automatic means of electrical protection against overwind is lacking. Thus, for cage lifts, not equipped with end switches, it is necessary to draw up the strands in the pile driver higher than the landing for the purpose of preventing the lift of the cage under the pulleys. In order to prevent the fall of the cage in the shaft in the case of its overwinding with a subsequent breakaway of the rope, one ought to install emergency catches.

Separate devices for overwinding have been used in a number of mines of the ore industry simultaneously with the use of the end switches. Given below is a brief discription of the enumerated devices to prevent overwinding.

The convergence of rail guides. The wedging of a lift vessel between rail guides is attained by converging the external surfaces of the rail guides. Convergence can be produced in the direction

shown on the diagrams a and b , Fig. 90. The reduction of the distance between the rail guides according to diagram b can be produced because of the cramming between the rail guides and the buntons or because of the increasing size of the rail guides.

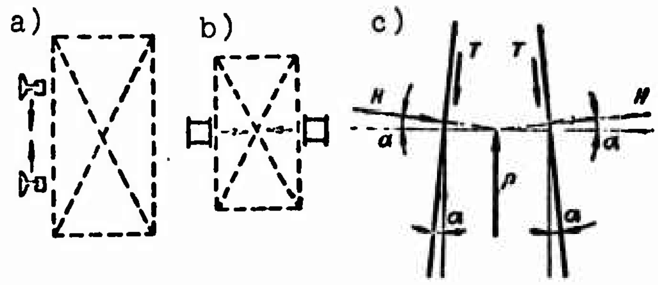

Fig. 90. Diagrams of the convergence of the rail guides: a) b) direction of the convergence of rail guides; c) diagram of the wedging of the lift vessel between the rail guides, strengthened by special designs, at the moment of the rupture of the rope.

Using a skip lift the convergence of the rail guides, as a rule, is not produced, but with a cage lift, one is usually limited to a convergence with an incline up to 1% with the magnitude of deviation in the rail guide within the limts of 30-50 mm.

With a tensile strength of the ropes of 100-200 t, the comparatively low rigidity of the rail guides and buntons and inclination of the wedging to 1% as a result of considerable sags, the effect of the wedging is small. Therefore, in certain cases a very simple reinforcement of the rail guides and buntons are made whereby for a section with the inclination of 1:100, double rail guides are installed, the buntons are somewhat reinforced, their spacing and so on shortened. Such measures increase the possible reinforcement during wedging of a vessel by 2-4 times and insure a more perceptive wedging with the convergence of the rail guides.

Convergence of rail guides, reinforced with special designs. During the wedging of a rigid lift vessel between conductors, reinforced with special designs (Fig. 90c), at the moment of rupture of the rope the following forces appear:

$$T = HK_{\text{тр}};$$
$$2H \sin\alpha + 2HK_{\text{тр}} \cos\alpha = P,$$

or

$$2H (\sin\alpha + K_{\text{тр}} \cos\alpha) = P,$$

where T – the force of friction; H – pressure on the rail guides during wedging; $K_{\text{тр}}$ – the coefficient of the friction of the slip; P – force in the lift rope.

By assuming

$$\sin\alpha + K_{\text{тр}} \cos\alpha \approx 0{,}5,$$

we will have

$$H \approx P.$$

Therefore, the upper theoretical limit of the value H is the tensile strength in the lift rope. The actual value H because of the elasticity of the cage, rail guides and of other designs will be substantially less. As a first approximation H can be taken to be equal to 0.5P; however, its value can be determined with any degree of accuracy allowing for the rigidity of the cage, rail guides and designs. Frequently, the designs, the reinforced rail guides, are calculated for the action of the force of thrust, equal to 50-70% of the tensile strength of the rope.

Figure 91 shows a section of the upper part of a single-hoist cage pile driver. Level *I* corresponds to the beginning of the convergence of the rail guides, and at level *II*, where the beginning of wedging of the vessel is expected, a powerful horizontal strap is provided. From level *II*, parallel to the rail guides, the elements of reinforcement which represent vertical metallic two-strip bars, are installed. The lower horizontal reactions of the bars are absorbed by the strap at level *II*. The upper horizontal reactions are absorbed by the connections at level *III*.

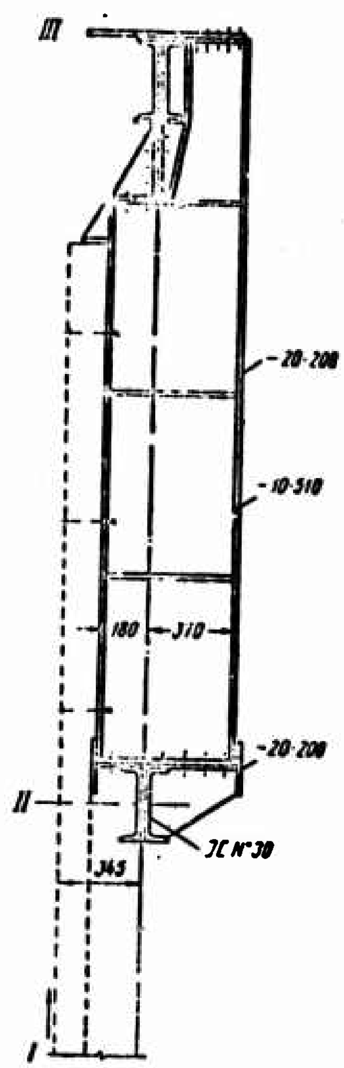

Fig. 91. A section of the upper part of a cage pile driver.

Figure 92 gives a plan and section of the upper part of the pile driver made from two cage separations. Here, just as in the previous example, *II* — level of the expected start of wedging of the vessels. Somewhat higher up the emergency catches are installed and still higher up, the stop bars.

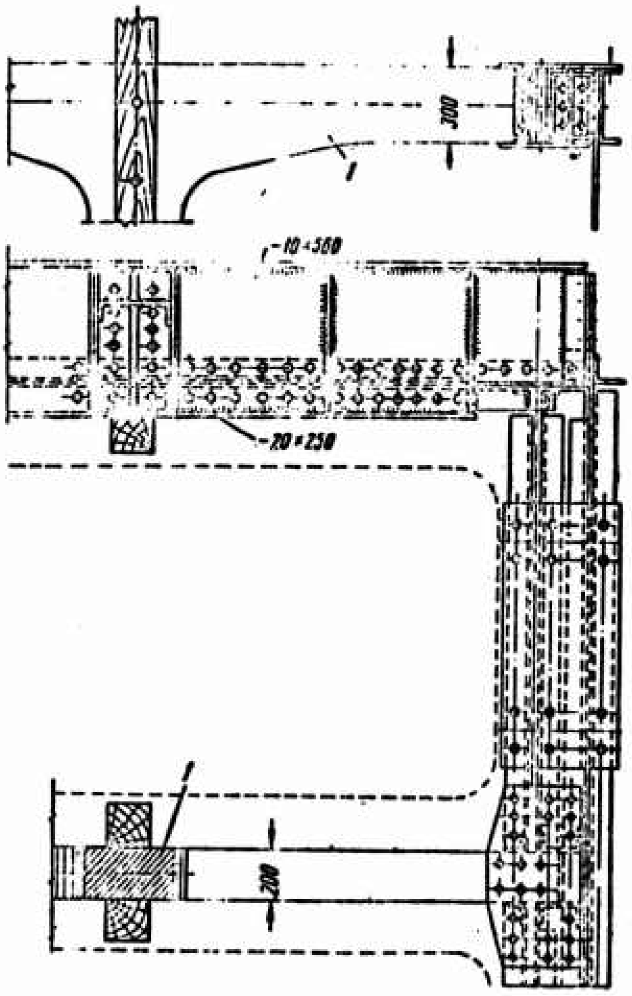

Fig. 92. The reinforcement of the buntons at the level of wedging of the lift vessel.

The outer rail guides in this case are reinforced similarly to that described above. The inner rail guides located at a distance of 200 mm in the clear, and the metallic double-T upright cannot fit within this overall size. Therefore, a T-shaped element

l made from cast steel is used here, which combines the reinforced bunton and simultaneously, the vertical element of reinforcement of the rail guides.

Safety lug-catch and stop bar device. Safety lugs are installed at the level of wedging of vessels and above them – stop bars. *The safety lugs* are installed taking into account the allowance for the minimum height of the fall of the cage to the level of the lugs after the break of the lift rope.

Figure 93 shows the *stop bar device,* which is fastened to the lower straps of the underpulley trusses. The sections of the straps are reinforced, the panels are shortened.

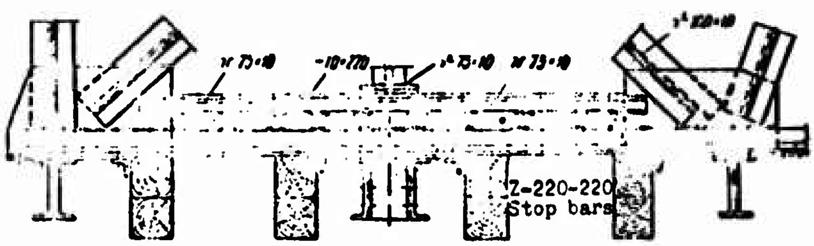

Fig. 93. The stop bar device in the pile driver machine.

The safety lugs and stop bar devices are given in Fig. 94. The bars transmit the loads to the welded beam, whose section corresponds to the loads and to the amount of span.

Analogous devices on overwinding are also used on wooden equipment; the reinforcement of rail guides in a section of convergence is carried out because of the close setting of the buntons. Furthermore, here the stop bars, reinforced with upright-spacers which transmit a part of the forces of the underpulley landing of the pile driver. One also installs the safety lugs, fixed to wooden underlug beams of the machine, on the section.

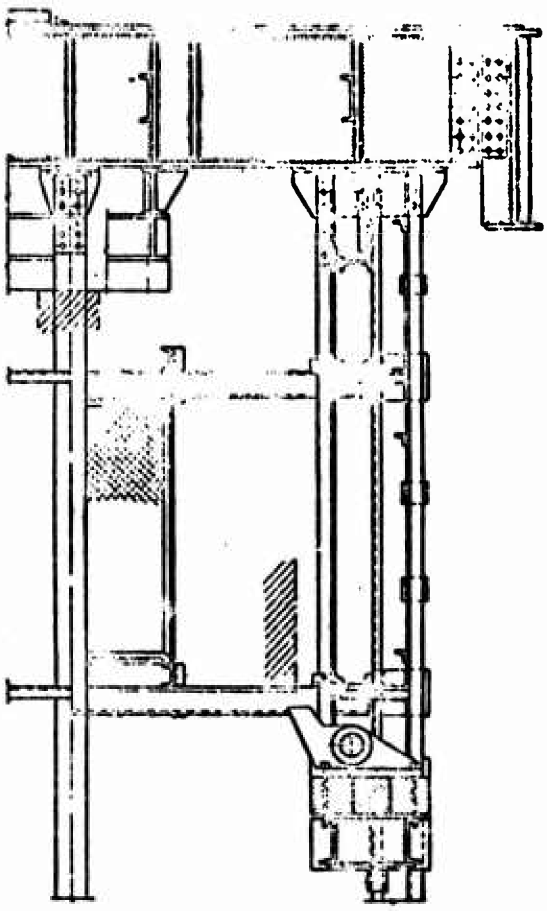

Fig. 94. The safety catches and stop bar devices in a frame pile driver.

The utilization of harpoons. At one of the English mine shafts metallic harpoons have been set on the pile driver; during overwinding the harpoons enter special boxes, fixed to the cage (Fig. 95); the latches of the harpoons prevent the fall of the cage. With end switches the Rules of Safety do not require the use of any mechanical measures of protection against overwinding.

Stop or rest bars fix the limiting point of the lift, serve as means to prevent damage during the lift of the vessels up to the guide pulleys and the safety element of the underpulley trusses. Thus, the bars are a form of protection to the pile driver. Their utilization has significance at a small height of overwinding by 6-8 (4-6) m for cages, and 2.5-4 for skips. At the greatest height of overwinding this device gradually loses its value.

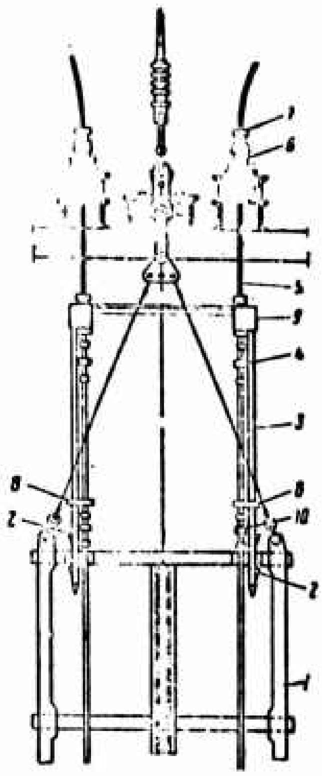

Fig. 95. Harpoon device: 1 – closet; 2 – latch; 3 – harpoon; 4 – brake shoes on the rope; 5 – rope (guide and brake); 6 – stop; 7 – support shoes; 8, 9 – fasteners for the harpoons; 10 – shoes.

The convergence of rail guides, simple or with reinforcement can be easily carried out with the bolstering of the machine; it applies only for cages at a limited height of overwinding – 6-8 (4-6) m. For cargo lifts the need to use such equipment diminishes.

In the absence of end switches on cage lifts based on the Rules of Safety the actual application of convergence of rail guides and of safety stop devices is necessary. However, without the stop bars the level of the lift vessel at the moment of rupture of the rope cannot be determined sufficiently accurately, as a consequence, the use of the safety catches loses its significance. Thus, for cage lifts, not possessing end switches, the *stop bar device is completely necessary*. For skip lifts, not possessing end switches, the stop bar device is also necessary.

The recommendations given above of cage pile drivers are entered in Table 23.

Table 23.

Designation of devices for overwinding	With end switches						Without end switches and under severe conditions of operation		
	requirement of the Rules of Safety	according to data from actual practice			recommended at a height of overwinding, m		requirement of the Rules of Safety	recommended at a height of overwinding m	
		in ferrous metallurgy	in non-ferrous metallurgy	in the coal industry	6—8 (4—6)	8—9 (6—7)		6—8 (4—6)	8—9 (6—7)
End switches	+	+	+	+	+	+	—		
Converging rail guides	—	±	—	±	—	—	+	+	+
Converging rail guides, reinforced by special designs	—	±	—	—	—	—	—	—	—
Safety lug-stops	—	±	+	—	—	—	+	+	+
Stop bars	—	±	+	±	+	+	—	+	+
Harpoons (with guide ropes) and analogous equipment	—	—	—	—	+	—	+	+	+

2. The Elements of the Pile Drivers

The overall sizes of the machine. During the determination of the overall sizes of the machine the required clearances between the advancing parts of the lift vessels and metallic buntons and spacers of the pile driver should be taken.

The least permissible clearances in the pile driver machine are given in Table 24.

One ought to point out that the clearances (without the brackets) are minimum and should be maintained during operation. This means that during the manufacture of the pile driver one should provide somewhat larger sizes of clearances.

Table 24.

Designation of the clearance	Minimum clearance, mm	Remarks
Between the lift vessels and the buntons, not supporting the rail guides	150 (160—180)	Construction sizes (in brackets) refer to the construction at a height of 20-60 m
Between two shifted lift vessels in the absence of a bunton	200	With rigid rail guides
Between the buntons and the parts of the lift vessels, separated from the axis of the rail guides at a distance up to 750 mm	40 (50—60)	Clearance between the approaching unloading roller and the bunton increases by 25 mm
The same, more than 750 mm	65 (80—100)	
Between the lift vessels and the shoring of the mouth or the designs of the pile driver	150 (160—180)	With the metallic and ferroconcrete designs and metallic reinforcement
The same	200 (200—250)	With wooden and ferroconcrete designs and wooden reinforcement with double-sided positioning of the rail guides. The larger construction sizes (in brackets) refer to ferroconcrete designs

The data given in Table 24 about the clearances between the buntons and parts of the lift vessels, separated from the axis of the rail guides to a distance to 750 mm, pertain, especially, to the clearances of the parachute equipment with cages.

A good deal of attention should be paid to the question of the purpose of structural clearances in the pile driving machine. An inadequate amount of clearances leads to the need to cut metallic designs and, therefore, compels one to apply local reinforcement to the elements of construction, which is not always possible because of the limit of the overall size of the machine. On the contrary, the purpose for excessively large clearances leads to an increase in the reinforcement in the elements of the machine, especially, in the buntons, receiving the load from the rail guides.

Figure 96 gives dimensional diagrams of certain pile driver machines.

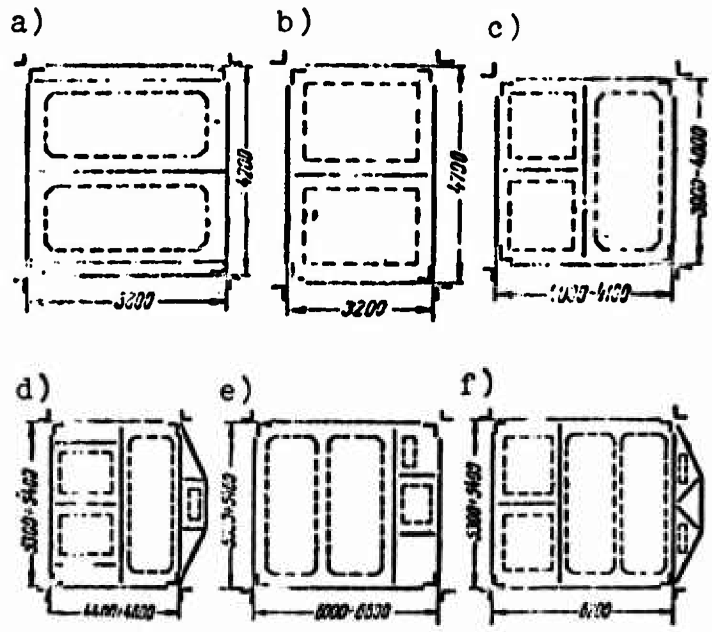

Fig. 96. The overall sizes of certain pile driver machines: a) with two cages for ore trams having a capacity of 1 m^3 (2.5 t): b) with two coal skips with a carrying capacity up to 9 t; c) two skips having a carrying capacity up to 8 t and a cage under the ore trolley having a capacity of 1 m^3, d) two skips having a carrying capacity of 10 t and a cage under the ore trolley having a capacity of 4 m^3 (10 t) with PTK type parachutes; e) two cages under the trolley having a capacity of 10 t and a cage of an auxiliary lift with PTK type parachutes; f) two skips having a carrying capacity up to 20 t and two double-deck cages under a trolley having a capacity of 10 t with PTK type parachutes.

Using a symmetrical position for the lift vessels, the pile driver equipment is substantially simplified: the number of various brands of installed machines in a mine shaft, as well as the expenditure of materials can be reduced; the structures can be easily unitized with repetition of units. Furthermore, using symmetrical positions of lift vessels, the rigidity of the machine and pile driver on the whole, other conditions being equal, will be maximum, and the location of the lift machines most compact.

The axial sizes of the machine by design can be obtained by means of summing up the overall sizes of the vessels in the clear, the width of the trusses of the machine and the clearances. With a width of the trusses of the machine at 200-250 mm, the tentative sizes of the machine by design in the axes of the uprights will amount to

$$a \times b = [a_1 + (500-600)][b_1 + (500-600)] \text{ мм},$$

where a; b — the axial sizes of the machine by design; a_1; b_1 — sides of the rectangle, which describes the contour of all lift vessels, located in a section of the mine shaft.

A size of 500 mm corresponds to a small and average size, a size of 600 mm – to average and large pile drivers. Upon the introduction of free buntons the sizes of the machine by design increase in one measurement by 250-400 mm.

Buntons. The distance between the tiers of buntons usually constitutes 2-3 m with wooden ones and 2.5-4.1 m (frequently 3.0 m) with metallic rail guides. In a number of cases the distance between the tiers of buntons doubles with the corresponding reinforcement of the rail guides.

Using a large quantity of buntons, a considerable part of which are the elements of the frames or of the trusses of the machine, it is frequently very difficult to hold to the required clearances because of the following circumstances.

The metallic design of the pile driver machine with its specifications for buntons carried out with a high degree of accuracy, can appear to be unsuitable because of inaccuracies, tolerated during bracing and reinforcing of the mine shaft, and during the manufacture and installation of the metallic pile driver structures, in turn, can introduce deviations from the designed sizes.

One should also keep in mind the possibility of accumulating errors for the sizes, and the need for correcting them within the limits of the pile driver machine. If the buntons are the elements of the machine, with the bracing made final during manufacture, then all the subsequent work for the correction of the allowable inaccuracies, operation for the necessary subsequent movement of the bunton are quite difficult. In such cases it is necessary to work under stressed conditions, following the installation of the structure, the cutting of the shelf buntons of the double-T beams, the removal of the beams with installations of new profiles outside the connections, and other laborious operations. The time involved in this case, is frequently very considerable.

The reduction and removal of the enumerated difficulties is assured by means of reducing the number of buntons by design and the creation of the possibility of their free movement to a certain degree by design.

The reduction in the number of buntons by design is attained by using one-sided bracing of the rail guides (Fig. 85b, and 97a). Undoubtedly, this solution should be connected with reinforcing the mine shaft. The possibility of the movement of buntons by design is assured by an appropriate design of the bracing of the bunton in the supports. Openings in the elements of the machine, which correspond to the openings on the supports of the bunton, should be bored during installation. Under plant operating conditions it is possible to provide temporary attachments for the bunton using two bolts in a normal position. In a number of cases the buntons can rest on the supporting part of the cross pieces of the machine and be bolted on or welded to the elements of the cross pieces. With an increase in the height of the pile driver and the number of buntons in the section of the machine the question of assuming some displacement of the buntons should be given considerable attention.

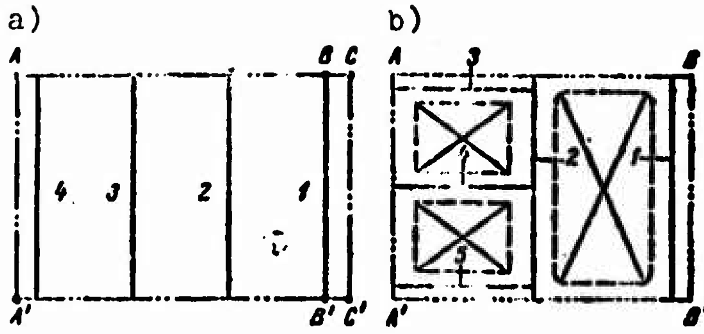

Fig. 97. Diagrams for the selection of the distribution of natural buntons, which allow certain shifts in the plain.

In general the following solution is recommended (Fig. 97a). If 1, 2, 3, 4 – buntons, and *AA'BB'* – the outline of the pile driver machine, then the matching of bunton 1 with the outline *BB'* of the machine is possible. The plane *BB'* of the machine in this instance should be installed with increased accuracy; the buntons 2, 3, 4 after a check of their positions are established with the small necessary displacement of their supporting parts lengthwise to the cross pieces of the machine.

It is also possible to use the free position of the buntons with the limits of the machine. In this instance (Fig. 97a) 1, 2, 3, 4 – buntons, *AA'*, *CC'* – the outline of the axes of the machine. This diagram, very convenient for installation and replacement of reinforcing elements, leads, however, to a small increase in expenditure of metal.

Figure 97b, shows the buntons 1, 2, 3, 4, 5 at another position of the lift vessels. With sufficient sizes of the machine it is possible to set the free buntons 1, 3 and 5. With limited sizes of the machine, buntons 3 and 5 are combined with planes *AB* and *A'B'* or bunton 1 with plane *BB'*. It goes without saying that in the last cases for planes, combined with the buntons, increased requirements should be met during manufacture and installation.

Applied and potential sections of metallic buntons are given in Fig. 98. Section *а* has the least transverse size, and section *б* the most convenient for bracing to the junction plates of the machine. However, both sections have a number of deficiencies, which should pertain to low strength and stability during the combined action of bending and torsion, and also during the combined action of bending in vertical and horizontal planes. Sections *д, к, н, у* and others are considerably more improved in the shown relationship.

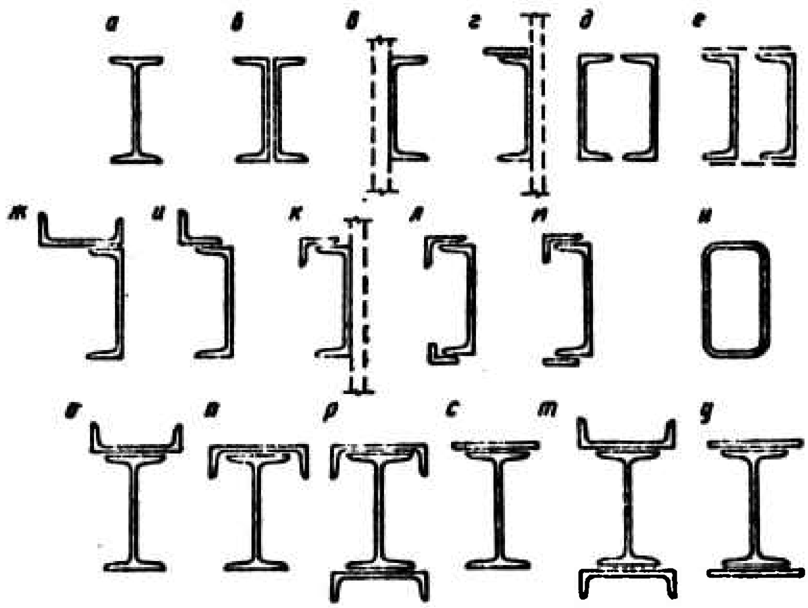

Fig. 98. Sections of metallic buntons.

One ought to show that for mine pile drivers in most cases there is no need to use buntons of streamlined designs. This can be explained by the fact that the supply of air passing into the shaft is predominantly through an underground duct, traveling past the pile driver.

Loads on the buntons. Vertical loads on the buntons are varied and depend upon the type of parachute devices.

Using the PDP type of parachutes and parachutes with longitudinal-cutting prongs of catches with wooden rail guides, and also using RKE and REK rail parachutes the loads which appear during the braking of the cage after it broke the lift rope, are absorbed by the rail guides and through them they are transmitted by the buntons. The amount of load on the bunton depends upon the amount of braking forces and the quantity of buntons, distributed by this load.

Apart from braking forces the bunton can also absorb a steady load from the weight of the plating and heat insulation of the pile driver, from its weight of the bunton, rail guides and other elements.

Using PTK and PKL type parachutes the braking forces are absorbed by the brake ropes and are not transmitted to the buntons. In this instance the vertical load of the bunton is only a constant load from the weight of the plating, heat insulation, its own weight of the bunton, rail guides and so on.

In most cases the vertical load is transmitted to the bunton eccentrically, causing, in this case the bending of the rod as well as twisting. The vertical loads of the rail guides and buntons during braking of the vessels are determined in accordance with the data, presented in Chapter V.

The vertical load, induced by the braking force, depends upon the length of the link of the rail guides and the distance between the tiers of the buntons.

The braking force is distributed to all buntons, to which one link of the rail guides is attached. However, the presence of a well made and operating rail guides on the compression and tension of the joints insures the participation of a number of buntons, located along the length of even several links of the rail guides in this operation. In this instance during practical calculations one assumes that with the calculation of the pliability of the

joints, the load is distributed to the buntons of two links of rail guides. One ought to note, that in each concrete case a load on the bunton can be specified by using the calculation of the rigidity of the buntons and conductors and the type of joints of the latter.

The horizontal load on the buntons, determined by pressure or by the rarefaction of air in the mine shaft or in a pile driver, usually does not exceed a value that corresponds to a pressure of 400 mm H_2O (0.4 t/m^2). This load is absorbed by the buntons, located on the outline of the machine. A windy load can increase the horizontal loads on the buntons up to 0.5 t/m^2 of its cargo area.

A horizontal load on the buntons also appears during the braking of the lift vessels, equipped with various parachutes. For the majority of contemporary parachutes the vibrations of the magnitude of of the braking force within limits up to 15% are inherent, which, however, is a high value for calculating the designs of the parachutes. It is possible, therefore to conditionally assume that the degree of change of forces in the rail guide, and also in the braking (or shock-proof) rope is equal to the smaller part of the above shown amount, which is equivalent to that amount assumed during braking using the difference in the forces in the brake ropes at a size, for example, of 2% of the overall vertical load ($6Q_{nop}$). One ought to further propose the possibility of a certain misalignment of the cage because of the large one-sided grooving of the wood or because of the one-sided pulling of the shock-proof rope. The arbitrary moment of the pair of so-called vertical forces with the arm of couple, equal to the mutual distance between the rail guides or between the brake ropes can be, in this case, absorbed only by the pair of the horizontally directed jet forces, applied to the shoes of the cages and equal to

$$\frac{2}{100} 6Q_{nop} \frac{b}{h},$$

where b — distance between the seizures of the parachutes by design; h — distance between the shoes or the rollers of the cage on a vertical line.

If we assume a value of $\frac{b}{h}$ equal ~0.5, then the horizontal force in the shoe of the cage, which appears during braking of the lift vessel, tentatively constitutes about 0.06 Q_{nop}.

In general it is possible to distribute this horizontal load, which appears during the braking of the cage, between the buntons, depending on the spacing and rigidity of the buntons and the rigidity of the rail guides. It can be approximately considered that the load is distributed over two or three buntons.

The above given approximate data about the distribution of the vertical and horizontal loads on the buntons, which appear during braking, are entered in Tables 25 and 26.

Table 25.

Designation of the loads	Amount of load on the buntons		Remarks
	using metallic and wooden rail guides and parachutes of the notched and PDP, RKE, REK types	using PTK parachutes	
Vertical loads on the buntons			
Constant load (weight of the plating, insulation, their intrinsic weight of the rail guides and of the buntons)	Based on the actual load	Based on the actual load	
Effect of the braking of the parachutes	$F_p = 6Q_{nop}$		

Table 25. (Cont'd).

Designation of the loads	Amount of load on the buntons		Remarks
	using metallic and wooden rail guides and parachutes of the notched and PDP, RKE, REK types	using PTK parachutes	
Horizontal loads on the buntons			
Pressure or the rarefaction of the air in the mine shaft	Based on the actual pressure, up to 0.4 t/m^2	Based on the actual pressure, up to 0.4 t/m^2	
Windy load	Based on the actual load	Based on the actual load	Usually insignificant and can be excluded from the calculations
Effect of the braking of the cage (for the majority of lifts)	$H \approx 0{,}02\, F_p \frac{b}{h} \approx$		b – distance between the seizures of the parachutes by design;
	$\approx 0{,}12\, Q_{пор} \frac{b}{h}$	$\sim 0{,}12\, Q_{пор} \frac{b}{h}$	h – distance between the claws of the cage on a vertical line;
The same, approximately	$H \approx 0{,}06\, Q_{пор}$	$H \approx 0{,}06\, Q_{пор}$	$Q_{пор}$ – weight of the cage with one man; F_p – the force of the braking of the cage.

Table 26.

Designation of the loads	Load on the bunton during the braking of the cage		
	using parachutes of the notched and PDP, RKE, REK types		using metallic rail guides and PTK parachutes
	using wooden rail guides	using metallic rail guides 12.5 m in length	
Distance between the tiers of buntons, m	2-3	4.1	—
The vertical load on one bunton during the braking of the cage	$(1.0-1.5)\,Q_{пор}$	$1.0\,Q_{пор}$	—
The same, with the realization of the calculation junctions of the rail guides (and with metallic rail guides of 25 m in length)	$(0.5-0.8)\,Q_{пор}$	$0.5Q_{пор}$	—
The same, with the introduction of connections	according to calculations	according to calculations	—
A tentative horizontal load on one bunton during the braking of the cage	$(0.02-0.03)Q_{пор}$	$(0.02-0.03)Q_{пор}$	$(0.02-0.03)Q_{пор}$

Rail guides. Used most frequently are the wooden and metallic rail guides. For wooden rail guides one ought to use the wood from hard and relatively rot-resistant species, especially, larch and oak. As an exception, it is permissible to use pine.

On sections, where the specific strength of the rail guides is necessary, for example, the unloadings landings of the lift vessels, local reinforcement of the rod of the rail guide is employed. On such sections the nonreinforced rail guides are worn out very rapidly and put out of order. In this case except for abrasion the cleaving of the wood, the fragments of the ribs of the rail guides, the failure of wood at the bolt openings and so on is observed here. The reinforcement, acting across the longitudinal axis of the rail guide, should be transmitted to the metallic design of the local reinforcement with the aid of reliable stops, which eliminate the crumpling and cleaving of the wood of the external and interior ribs of the rail guide and bolts. The replacement of the rail guides in such places should be easy to do.

The reinforcement of the rail guides amounts to the introduction of channel bars and other profiles in the individual sections.

As metallic rail guides one usually uses rails of type P38 and P50 having a length of 12.5 and 25 m. The rail guides are fastened to the buntons with help of metallic clamps and bolts.

Clearance between the guide shoes of the lift vessels and metallic rail guides is taken as 5 mm, but with wooden rail guides, 10 mm on each side.

Along with rigid rail guides rope guides are presently used. The motion of the lift vessels with rope guides occurs with a certain deviation in the vessels by design; therefore, between the lift vessels in the shaft stop ropes are usually installed, which prevent the possibility of collision.

The advantages of rope rail guides: the possibility of a deviation in the claws of the lift vessel from the rail guides, the decrease in resistance to the motion of the vessels, the possibility of rapid installation and replacement of rail guides are excluded.

The deficiencies – the considerable quantity of rope, the large clearances and the enlarged sections of the shaft and pile driver machine, the need for the complete replacement of the rope in the presence of a single local defect, the need for landing equipment for bracing the ropes on the pile driver during considerable loads on the supporting structures. At one of the ore mine shafts with two cage lifts using counterbalances, the number of rope guides in the shaft constitutes 14 pcs; furthermore, six stop ropes are set up, all of them in the shaft section and the pile driver machine has 20 ropes. In accordance with this 20 end bracings of the ropes are provided on the pile driver. The latter are usually combined with screw jacks, equipped with gauged springs, which in the presence of control, approximately assures constant values of tension in the ropes. Loads from the tension of ropes usually constitute 1-2 t for every 100 m of length of rope.

Planking and hermetic sealing of the machine, the windlass equipment. The pile driver mounting usually should be plated with continuous or latticed planking. Continuous planking is used beyond the limits of the deckhouse building or inside the latter; one installs the lattice planking inside the deckhouse building. As continuous planking for machines corrugated or flat metallic, and especially, aluminum, sheets are used (Fig. 99a, b, c), and sometimes corrugated asbofaner* and sectional ferroconcrete slabs. In separate exceptional cases the machine guard can be made of monolithic ferroconcrete.

In wooden pile drivers and in pile drivers with short period services, installed over the shafts with an outgoing ventilation jet, planking is used, made from asbestos-cement sheets or wooden planks.

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*[Translator's note: The term "asbofaner" cannot be found in available sources, but its two roots imply asbestos-veneer].

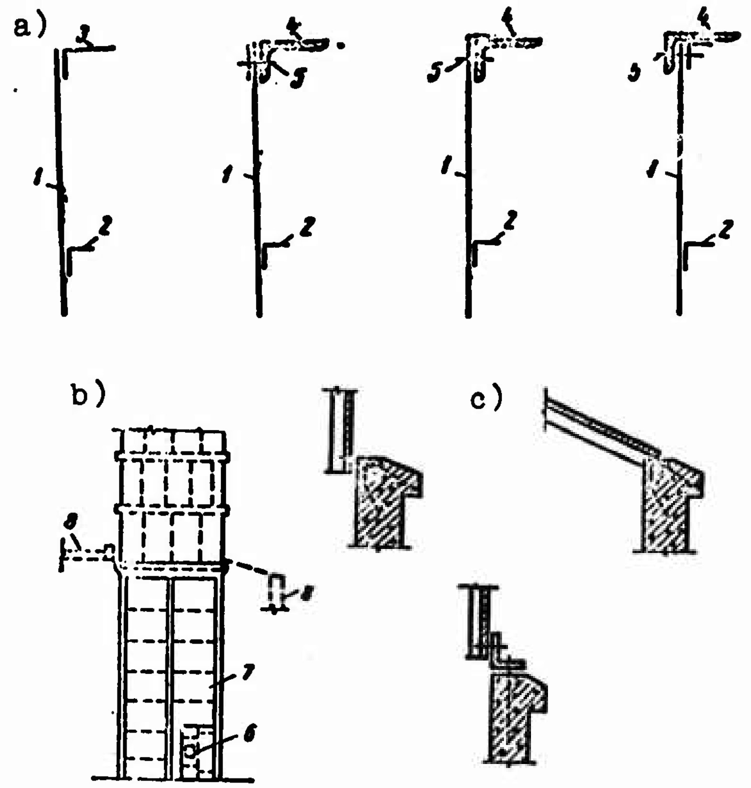

Fig. 99. Planking and the hermetic sealing of a machine: a) the variations of planking of a machine using aluminium sheets; b) diagram of a pile driver machine with hermetic planking and windlasses; c) the union of the planking of the pile driver and of designs of a deckhouse building; 1 – aluminium sheet; 2 – auxiliary aluminium corners or bands, joined to sheet 1 by contact welding; 3 – braced aluminium corner, joined to sheet 1 by contact welding; 4 – braced steel corner stained in two layers of aluminium paint along the entire perimeter of the section and joined with the aluminium design by bolts; 5 – bolt; 6 – control opening; 7 – hermetic windlasses; 8 – designs of deckhouse building adjacent to the machine.

If heat insulation is necessary the planking of the pile driver machine is usually made from a mineral fiber or mineralized slabs, placed between two metallic, sheets especially aluminium, between the ferroconcrete sheets of the guard and the external planking made from aluminium or lightened ferroconcrete slabs, between asbestos-cement sheets, and so on.

For temporary structures with an exhaust air jet, the use of boards having a thickness of 50-100 mm, protected by asbestos-cement, sometimes metallic sheets, and others, worm planking.

In the construction of capital mine shafts everyone to a greater degree uses warm planking for machines. Furthermore, in a number of cases they also produce supplemental heating of the inner space of machines with the aid of heating devices.

One should consider that the guards of the machines are in contact with the moist atmosphere, which is characterized (because of the small volume of the machine) by variable temperatures. During the replacement and repairs of rail guides the elements of the guard are subjected to impact and various mechanical actions. Therefore, the designs of the guard should differ by relative stability, which corresponds to severe conditions of operation over prolonged periods of time.

The duration of work in the construction of a pile driver (including the installation of the machine and of reinforcement, the revetments of the machine to the deckhouse building) is of great significance when putting a mine shaft into operation. Therefore, when selecting this or another type of planking for the pile driver one ought to pay special attention to the simplicity of the equipment, to the possibility of procuring heavy panels or shields by means of specified orders, by rapidity of installation of the heavy parts of the planking.

The hermetic sealing of the machine is attained by means of introducing compact metallic sheating or ferroconcrete in the seams of the walls.

Figure 99b, depicts a diagram of a metallic machine, which possesses a continuous hermetic planking. The hermetic welded planking is made from sheets, reinforced with ribs, since larger consolidated sheets are more convenient. At the intersections of

the lift ropes with the planking, gates are set up using wooden shields, which are closely spaced near the ropes made of leather and other materials.

The deckhouse building or a part of it should also be airproof; for the distribution of the trams beyond the limits of the deckhouse building locks should be set up. In this instance airtightness of the machine at its unions with the elements of a deckhouse building should be provided which is attained by introducing matching corners or other profiles, built-in using anchors in ferroconcrete designs along the outline of the abutting parts adjacent to the machine of the deckhouse building (Fig. 99c).

The windlass in the pile driver machines – usually metallic – are recoil, lift and flap types. Used more frequently are the recoil windlasses, which are most convenient as a safety device, interlinked with the lift. One ought to consider that with two single-cage lifts, the windlass they can be merely extended to the different sides. A hermetically sealed windlass should have a frame, calculated for a lateral pressure up to 0.4 t/m^2. The thickness of the sheathing of the windlasses is 3 mm. The windlass is equipped with a small door with a small control which shuts the aperture, and which serves to equalize the pressures before opening the small door. When using this device on the windlasses, one should pay special attention to the density of the sheathing and the abutting of the double sections of the small door and windlasses to the sections of closed sheathing, where it is necessary to assure compactness. For this purpose one can apply a sealing tubular rubber padding.

Trusses and the frame of the machine. By design, a machine usually has a right angled section, divided into several compartments depending on a quantity and position of the lift vessels.

In pile driver machines the elements of the machine sustain the considerable forces. The flat trusses, located on the perimeter of the machine, are symmetrically developed relative to the local axes of rod designs. The following lattices of trusses are used: semidiagonal (Fig. 100a, b) which possesses the greatest number of joints; crossbracing with additional cross pieces (Fig. 100c), which represents a combination of the various semidiagonal panels and which differ by a fewer number of joints in comparison with semidiagonal lattices; diagonal, (Fig. 100d), which is characterized by relatively long paths of directional forces in the rods of the trusses; cross (Fig. 100e), which is a combination of two diagonal lattices. It is customary to assume that these or other angle braces of a given lattice operate under tension, and the cross pieces, under compression.

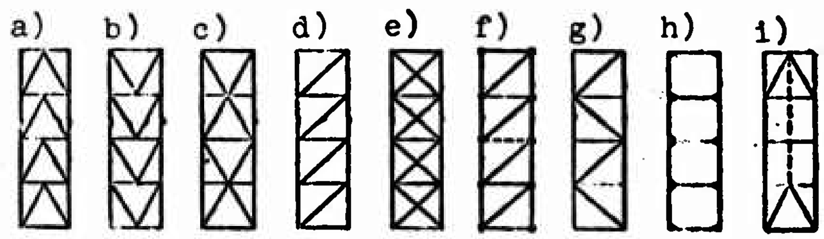

Fig. 100. The lattices of machine trusses.

In pile driver machines some, and especially diagonal lattices, have deficiencies. Figure 100 f, depicts a diagram of the lattice with cross piece-uprights, supporting simultaneously the function of buntons. The cross pieces are included in the composition of a truss as necessary elements of the latter. The forces in these trusses subsequently pass through the angular braces and cross piece-buntons. The basic purpose of the buntons – the fastening of the rigid rail guides. In a number of cases the forces of the braking of the vessels are transmitted from the rail guides to the buntons. In this case the buntons undergo considerable loads. In individual cases of an emergency nature, the loads can be concentrated on any bunton of a pile driver and these loads which exceed the calculated loads by several times, as a consequence, put

the cross piece-bunton out of order. Thus, local deformation of the bunton can lead to serious consequences. Therefore, the combination of functions of the bunton in one element and the required rod of the truss of the structure should not be considered as a better solution (this observation does not refer to the positional free buntons at one level with the cross pieces of a truss).

For the formation of machines one ought to use trusses with a triangular-shaped lattice (Fig. 100j), cross-bracing with additional cross piece-upright (Fig. 100c), sometimes simple cross-bracing.

The diagram of the lattice of a frame machine is represented in Fig. 100i. Using a considerable length of the cross pieces of a machine and large loads, the partial utilization of semidiagonal is possible in separate panels with cross pieces, connected by lightweight suspensions (Fig. 100k), and which relieve the light frame lattice from the action of the emergency loads of the buntons, also give it wider application. Frame lattices are usually used in pile drivers with the small values of transverse forces on the machine mounting.

With wide machines in the plane of the lift sometimes it is expedient to use a frame-lattice design of the machine.

Buntons should not be included in the composition of the elements of the lattice of trusses of a machine, thereby insuring the installation and the dismantling of each bunton, individually. It is also necessary to create the possibility of subsequent correction of the position of the buntons by design which will allow one to compensate for the possible deviations in the actual sizes from the designed ones.

Given above, in fact are those designs for machines of diagonal pile drivers, which in view of the small loads, are made lighter and in a number of cases, can be designed as framed.

Frame panels. At a level of the landing for a machine (in the plane of the main trusses or transverse planes) frame panels are arranged, whose sizes are determined by the sizes of the trams and of the skips. The sizes of the apertures for the skips are determined by the geometrical construction during their motion along the unloading curve.

The sizes of the apertures for the haulage of the trams can also be determined by the overall sizes. The height of the aperture, based on the convenience for landing the people, is taken as not less than 2.2-2.5 m in the clear, calculated from the level of the head of the rails of the landing to the cross piece of the machine, which is limited by aperture.

On the lower landing, where the lift vessels (cages, skips) enter through the apertures of the machine, the height of the apertures is determined by means of constructing overall size outlines of the vessels during their initial motion. For the convenience of starting the motion of the cages it is expedient to assume that the height of the aperture is somewhat greater than the height of the cage which is especially important for cages having great length. The uprights of the machine, which do not limit the size of the aperture when only using a device of free buntons, do limit the development of the width of the apertures.

Through the auxiliary shafts the descent of linear-shaped objects is made: timbers, rails and pipes. The size of the aperture of the machine in such cases should be verified in relation to the convenience of starting the motion of the linear-shaped objects during their descent.

The height of the aperture for the descent of the rails of a normal track depends upon the magnitude of the critical size of the machine by design: with the size of the machine by design at 3-4 m the necessary height of the aperture for inserting the rails of a normal track comprises 6-8 m. For the insertion of narrow-gauge rails and linear-shaped timber (length of 8.5-8.8 m) the height of the aperture of the machine should be not less than 5-5.5 m.

If one puts the auxiliary shaft into operation later than the main shaft, then the pile driver machine of the main shaft should have apertures, which insure the entry of objects of a length of not less than 8.8 m into the mine shaft. In this instance the height of the aperture of the machine in the clear at the depth of the compartment of 3-4 m, constitutes 5.0-5.5 m, and the corresponding full height of the frame panel of the pile driver is 6.5-7.0 m.

The frame panels for the unloadings of skips, usually located in transverse planes, have a comparatively small width, which exceeds by 2 m in most cases the height of the panels within the limits of 6-12 m.

In each concrete case the sizes of the apertures should be taken along with the calculation of the technological requirements.

The design of frame panels is shown in Fig. 101. Usually, the frame panel is made basically because of the reinforcement of the uprights within the limits of the frame and neighboring panels, meeting the need of whatever arises in the equipment of especially rigid cross pieces; the rigidity of the cross piece is secured because of the attachment of the rods of the adjacent panels of a truss to the reinforced bars of the machine.

The skip frame panels of the pile driver machine, located at a height which corresponds to the position of unloading the skip bunkers, constitutes the part of the unloading skip equipment, the basis of which are the unloading cantilevers, composed of cantilever

trusses of the machine and the transverse joints, located in planes passing through the external rods of the cantilever trusses.

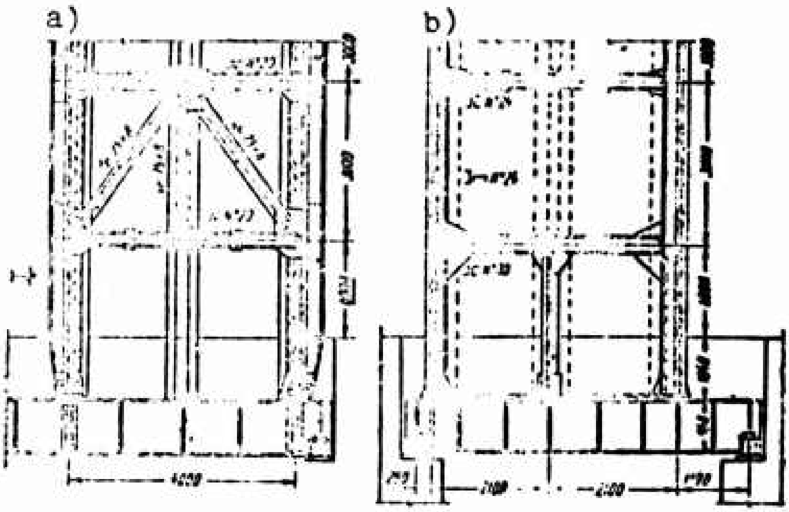

Fig. 101. Frame panels of a machine: a) with a lattice of the machine; b) with a frame of the machine.

Detachable panels. To avoid considerable sizes of frame panels of a pile driver machine sometimes so-called detachable elements of machine are used. In these cases a large frame panel is usually separated by the height into two parts. The upper part is filled with braces and cross pieces, fixed to the main junction plate with the help of bolts.

Through the frame panel the haulage and supply of items of limited length is done. With the entry of lift vessels into the machine, and also upon an entry undersize linear-shaped items the work of a lift device stops, and the rods, fill the upper panel are removed and reset, when the necessity for which an increase in the height of the aperture diminishes. The extensive use of detachable panels is not recommended because once the cross pieces and braces are removed, in practice they are not always installed in a place when there is less need for an increased aperture, and the frame panel should have the minimum required height. The elements of the frame should be rather rigid; in a special case, the reduction in the

magnitude of the bending moments in the uprights of the frame can be achieved because of the rigid closing of the uprights of the frame at their base. This method is more reliable than the utilization of detachable panels.

A pile driver should be checked out under all acting forces based on the assumption that there is an actual frame panel, the height of which should be taken in accordance with the given indications.

Where the machine is partly the designs of the construction, supporting considerable loads, that which is proposed for detachable panels has special value pile driver machines. The increase in the height of the frame panel of the machine, whose stability assures the bracing of the elements of the pile driver, has comparatively little effect on the amount of reinforcement and on the sections of the elements of the machine. The absolute sizes of the sections of the rods under similar conditions are small, and that is why the height of the frame panel can also be accepted as practically any quantity.

Undercatch beams and supporting frames of the machine. The uprights of the machine are usually installed on the underside pile driver beams, which are connected to one general supporting frame (Fig. 102).

When selecting the diagrams of the location of underside pile driver beams, one ought to consider the diagram of the pile driver and the condition of the support of the support of the beams. With rigid junctions of the uprights of a machine with the underside pile driver beams and with the appearance in this instance of the joint bending moments, it is expedient to install underside pile driver beams parallel to the plane of the main frame of the pile driver. With a hinged support for the upright of the machine such a position for the beams is not required. It follows, however, to take into account, that the hinges, using conventional riveting,

possess considerable rigidity which also leads to the appearance of bending moments in the supporting joints of the uprights. Thus, also in this instance, it is advisable to locate the underside pile driver beams parallel to the plane of the main frame of the pile driver.

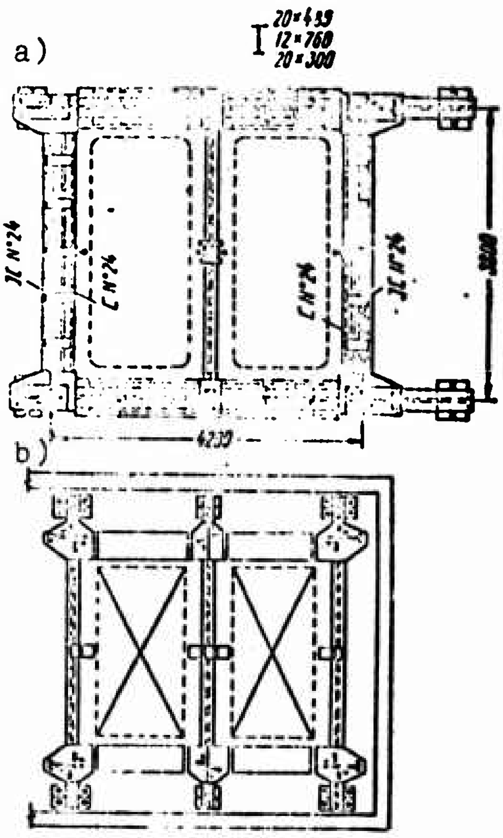

Fig. 102. The supporting frames of the pile drivers.

Using the support underside pile driver beams on separately standing foundations, it is quite important to take measures for the maximum reduction of the span of the beams. In this instance the transverse position of the underside pile driver beams is most expedient, if sufficient stability of the beams is provided for.

When selecting the means for positioning the underside pile driver beams the direction of the haulage, the presence of the stops at a level lower than the landing, and also the sizes of the transverse section of the mouth of the mine shaft have great value.

With the most extensively used haulage at mines, normal to the plane of the lift (Fig. 102a), the underside pile driver beams are usually parallel to the planes of the lift, i.e., they are found in the planes of the main trusses or frames of the pile driver. Using the stops on a lower landing the understop beams are frequently combined with the underside pile driver beams.

With haulage parallel to the plane of the lift, and with a transverse position of the underside pile driver beams a combination of underside pile driver and understop beams is also possible. Sometimes, with the same haulage and favorable outlines of the collar a scheme with the location of the underside pile driver beams in the planes of the main trusses of the pile driver along with transverse understop beams, is used. The scheme is also entirely applicable with the absence of stops on the lower landing.

With haulage normal to the plane of the lift in certain cases the scheme in Fig. 102b, is used along with a transverse position of the underside pile driver beams and longitudinal understop beams.

The sections of the understop beams, combined with the underside pile driver beams are given in Fig. 103. Inasmuch as the understop beams undergo large dynamic loads from the landing of the cages, the utilization of welded designs is allowed with good shock absorbers under the stops, whereby wooden bars, are used, located above the understop beams.

With an installation of separate understop beams, not connected to other designs for which a place is made on the upper landings, sections are used, composed predominantly of channel bars based on the type of section, shown in Fig. 103b, but in a welded finished piece, the sections are based on those in Fig. 103d, and others.

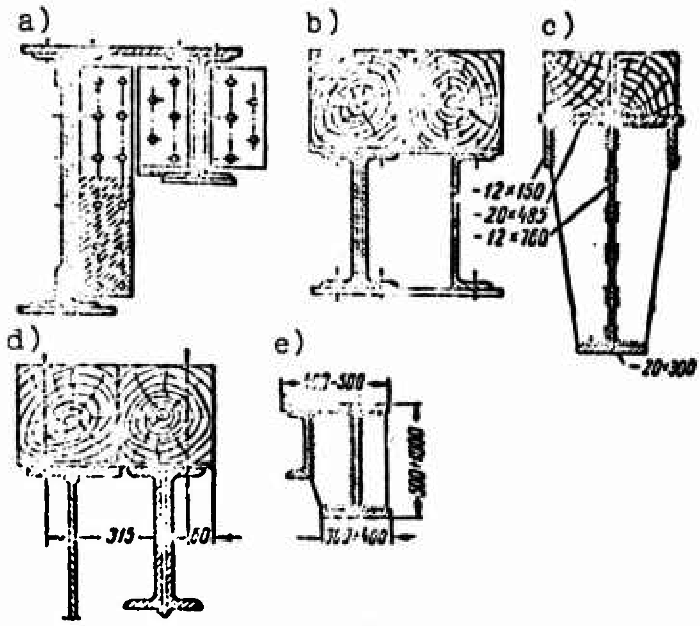

Fig. 103. Sections of the understop beams, combined with the underside pile driver beams: a), b) riveted; c), d), e) welded.

The support of pile drivers. The forces in the rods of pile drivers are transmitted to the supports predominantly through the bracing and underside pile driver beams, supporting the machine.

With the location of the underside pile driver beams parallel to the axis of the lift, used for the cage of the pile drivers with haulage perpendicular to the main truss or frame of the pile driver, the underside pile driver beams are also simultaneously understop beams. With haulage parallel to the plane of the lift, the underside pile driver beams are frequently perpendicular to the plane of the lift and are also simultaneously the understop beams. In all cases the trend is towards the largest possible reduction in the spans of the beams.

The underside pile driver beams should possess a high degree of rigidity. The sag of the metallic beams should not exceed 1/600 of the span, the height of the beams of pile driver machines constitutes 1/6-1/8 of the span; moreover the underside pile driver beams are conveniently arranged in such a way that the displacements of the top of the pile driver with sag of the beams would be the least. Specifically, the supporting joints of the beam is conveniently located symmetrically relative to the machine.

Underside pile driver beams of diagonal pile drivers, which have lighter machines and which undergo action of lesser loads, are made somewhat smaller in size and weight. If a diagonal pile driver has only a transverse frame machine, then the improved lower cross piece of the latter can simultaneously serve also as a underside pile driver beam.

The top of the underside pile driver beams are located lower than the top of the collar by 0.8-2.0 m in the cage pile drivers by 0.2-0.4 m in skip pile drivers.

Usually, the machine rest, the underside pile driver beams, and the bracing on the separately standing foundations. In individual cases the machines of the pile drivers rest on special foundations, located at a certain distance from the outer contours of the shoring of the mouth of the mine shaft. Such a solution is necessary, if there is danger of a rock fall during the driving of the mine shaft, with rigid ferroconcrete pile drivers and with the inadmissibility of the sags of the machines; with larger, unique loads and in other cases.

One ought to take into account that the support of the machine on special foundations is undesirable in connection with the need for substantially moving the machine or machine frame on the foundations. Furthermore, in a mine shaft a large number of various devices, foundations and ducts are located which impose a severe limitation on the spacings based on the size of the landings adjacent to the mine shaft. Therefore, the placing of the special foundations of a machine or of a machine frame creates a number of additional difficulties.

With A-frame pile drivers, the foundations of the latter are separated from the shoring of shafts, and the machines, as a rule, are suspended from the elements of the pile drivers.

In the iron-ore industry one additional solution is also used, the essence of which consists of the unitizing of the shoring of the shaft, the foundations of the pile driver and the foundations of the deckhouse building into an entity with the location of all shown construction on a general ferroconcrete foundation slab. In this way, on one hand, the simplicity of the construction work is assured, and on the other, a relatively low actual pressure on the ground under the foundation, is assured. From this point of view of a guarantee against possible sediment of the pile driver machine the described solution is acceptable because the presence of even significant rock falls during the driving of mine shaft appears to be slight just as for settlement, but there is an increase in the actual ground pressure under the foundation slab, the overall measurements of which are quite considerable.

Booms. In order to insure stability of the booms in the planes of the main frames of the pile driver, in a number of cases joints are used between the boom and the machine, installed in order to reduce the length of the booms of pile driver machine.

In transverse planes various basic schemes of booms are used. For pile drivers the transverse stability of which is assured by sufficient rigidity of the machine of a small height (not exceedingly 20-25 m) or by other means, the boom has the simplest rectangular form (Fig. 104a). Frequently, a boom of the machine and diagonal rigid pile drivers has a trapeziform form (Fig. 104 b). The width of the boom at its top is equal to the width of the pile driver machine, and at the base it is equal to the value B, specified from the condition of providing transverse stability of the construction. A check of the stability is produced under the assumption of an inversion of a pile driver relative to axes 1-1 and 2-2 (Fig. 104c).

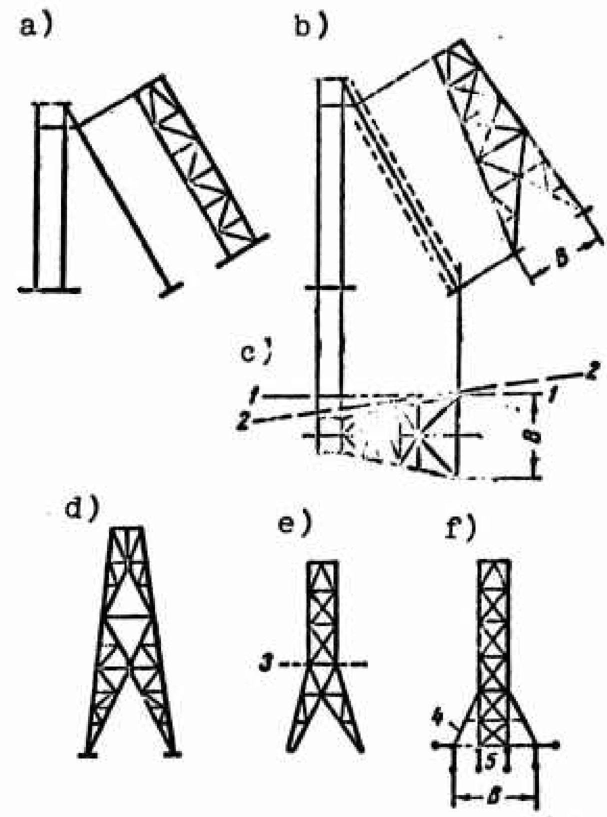

Fig. 104. Diagrams of the trusses of the booms; a) with parallel straps or a corresponding frame; b) trapeziform (the boom of the diagonal pile driver is shown by a dotted line); c) diagram for checking the pile driver by inversion; d) member of the boom along the axis of symmetry; e) with parallel straps in the upper part and with span uprights of straps at the base; f) booms of the pile drivers of small and average height.

Allowing for loads of its own weights, which assure the stability of the construction taking into account the anchors, with a coefficient of 0.9-0.8, it is possible to make a calculation of the stability from the method of calculating the limiting states. In this case, the moment which turns the construction over, determined with the advent of the coefficients of overload, should be somewhat less than the retaining moment, determined in accordance with the recommendations, given above.

The selection of the value of the span of the boom *B* is usually made based on the equalities calculated according to normative loads of overturning and retaining moments since one assumes the retaining moment as a coefficient of 0.95.

The span of a boom at its base *B* constitutes 0.3-0.35 of the height of the pile driver. Thus, even at the average height of the pile driver of 40 m, the width of the boom may be, for example, equal to 12-14 m which characterizes this part of the pile driver as an immobile transporting unit. With the partitioning of the boom along the axis of symmetry (Fig. 104d) and with a corresponding division lengthwise, the width of the starting mark is shortened two times. However, even in this instance, the sizes of the semitruss of the beam, as a rule, exceed the permissible overall size from the conditions of railroad transportation. This circumstance frequently creates the need to transport the boom from the plant to the assembly place for dismantling the kinds of elements.

Under the condition of disconnecting the boom in the plane of the main frame of the pile driver, for the average height a boom with parallel straps around the upper part and with the span of the uprights at the base, lower than section 3 (Fig. 104e) is convenient – the joint of the cantilever with connections in the plane of the main frame.

For pile drivers of small and average height booms basedoon the type shown in Fig. 104f, can also be used. The straps of this boom are parallel over the entire length and are located at a distance equal to the width of the machine. In a lower part the boom is provided with a bracing strap 4, which together with the lower strap 5, forms a simple rigid truss, which transmits forces to the supports.

The bracing strap 4 can be easily separated from the boom during transport; therefore such a boom is convenient to manufacture and transport. The structural lattices of this type can be expediently employed rather extensively in the transverse plane of the pile driver machine.

For the pile drivers of great height it is expedient to segment the upright straps of the height, form lattice rafter of the boom 1 (Fig. 105a). Points A, Б, B represent here the three supports of the pile driver. This diagram was recently used for several mine pile drivers having a height of more than 50 m. A certain modification of that described above, for the diagram of the boom is presented in Fig. 105b. For high pile drivers the utilization of tension, which assures the stability of the pile driver in a transverse direction is expedient. In this instance the boom can be plane with the parallel straps (Fig. 105d). All diagrams of booms, shown in Fig. 105, are especially expedient for high machines and diagonal pile drivers.

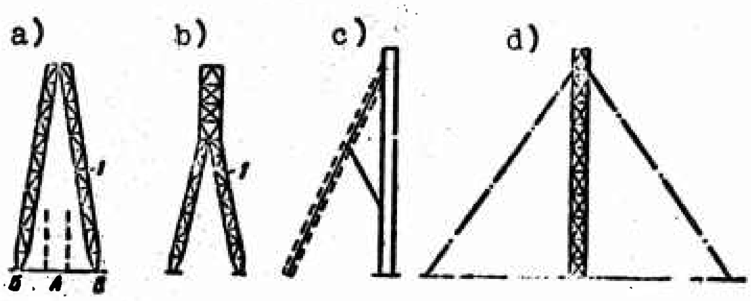

Fig. 105. Diagrams of booms of high pile drivers.

Using the most popular solutions of booms of average and high pile drivers according to height booms are frequently employed (Fig. 106a), which are installed horizontally, i.e., normally to the machine (position 1), normally to the boom (position 2) and at an acute angle to the boom (position 3). In the joint *A* of the intersection of the bracing with the boom a vertical force *P*, consisting of about 50% of the weight of boom is applied.

By breaking down force *P* into a number of components 1 and *I*, 2 and *II*, 3 and *III* (Fig. 106b), it is easy to establish that vectors *A*1, *A*2, *A*3 are close in value, the spans of the struts, 1, 2, 3 are also equal. Thus, the distinctions in the solutions of angle braces consist predominantly of the different actions on

the pile driver machine and boom. Bracing 1, and to a lesser degree, bracing 2, are used near the center of the machine and small pressure forces appear at the center. Bracing 2 hardly causes a bend in the machine and assures the greatest rigidity of boom and the absence of any kind of displacement of point *A* in direction 2 during the deformation of the boom.

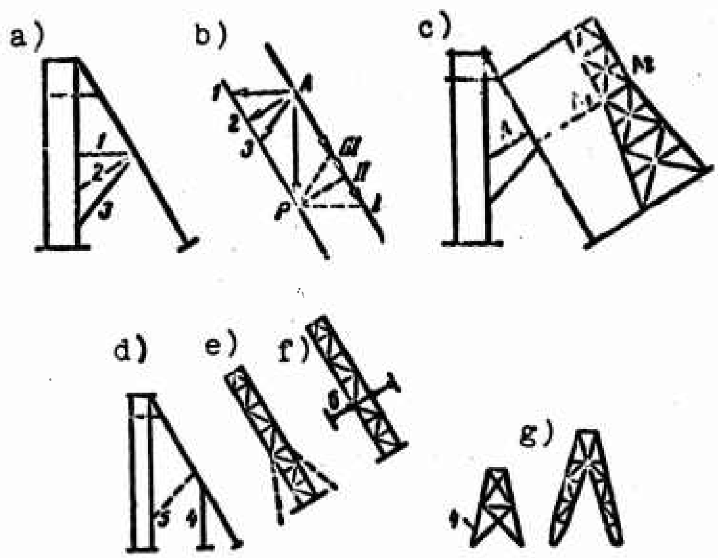

Fig. 106. Bracings of the boom: a), b) the diagram of the forces in the bracings and straps of the boom with different positions of the struts; c) boom with parallel straps of upper part and broadening at the base of the boom; d) the uprights of the boom; e), f) diagram of the boom crosswise; g) the span of the auxiliary uprights of the boom at the bases; 1 – horizontal bracing; 2 – bracing, set normal to the boom; 3 – bracing, set at a certain angle to the boom; 4 – auxiliary upright; 5 – bracing; 6 – a joint.

With a decrease in the rigidity of the machine the recommendation given here takes on considerable significance. With high and lightweight machines in a number of cases it is expedient to make a check on magnitude of transverse displacements of point *A* in the directions 1 and 3, and to determine the corresponding values of forces; bracing 2 or bracing 3 close to them are recommended. It is not necessary to give specific attention to the expenditure of material for the manufacture of bracing 2 and 3, themselves. The

absolute values of the forces in these rods and the expenditure of material for them are small (the sections of the rods are determined according to flexibility).

Frequently, a boom is made with parallel straps in its upper part up to point *A*, i.e., up to the cantilever of this or another bracing (Fig. 106c). The formation of joints of type *A*1 and *A*2 is possible and convenient, namely in the presence of bracings, lying in the plane of the main truss of the pile driver and directed towards point *A*.

The utilization of the auxiliary supports (upright) of the bracings merits attention, the length and section which hardly differ from the length and section of the bracings. Thus, Fig. 106d shows uprights 4, setup instead of bracing 5. The uprights are laid out crosswise at a base (Fig. 106g); and together with the grating they represent a trapeziform flat truss. The latter sustains the vertical loads of the boom and insures the work of boom in the main plane as a double-span beam. In a transverse the presence of truss 4 is equivalent to the creation of additional connections 6 near the center of the boom (Fig. 106f). With relatively small investments of material on truss 4 its presence insures the completion of the boom as a truss or of frame with parallel straps, with uniform panels and joints. Such booms work well for all types of basic loads. Thus, the basic pressure forces produce a monotonic spans by the support. Under the indicated conditions the weight of the boom is comparatively small, and the conditions of manufacture, transport and installation of the boom differ by having the greatest conveniences.

The introduction of auxiliary supports of the boom creates the possibility of utilizing the booms with parallel straps for all

average and some large pile drivers. Depending on transverse sizes and sections of the elements of the machine the utilization of the flat bracing with uprights is possible in a number of cases for pile drivers with a height up to 40-50 m. The booms with parallel straps and intermediate transverse frames (auxiliary supports) simplifies the task of the typification of deckhouse pile drivers.

The booms of pile driver machines have comparatively small sections in the plane of a main frame of the section. Their stability is secured by the bracing, directed from the machine. On the other hand, at the cantilever site towards the head of the pile driver it is difficult to develop a boom in the plane of the lift.

For the transmission of loads from the pulleys to the boom usually improved under pulley trusses are developed to the horizontal. This does not pertain to booms of pile drivers of the diagonal system, the section of the straps of which are reinforced with an increase in a load and length, respectively, and whose branches are spreadout.

The span of the branches of the boom in the plane of the main frame of a diagonal pile driver also assures a more convenient support of the underpulley trusses, and the section of the straps of booms in this instance can be made in the shape of a cross or a tee from the two corners (Fig. 107a). In necessary cases it is possible to form a section of the strap from channel bars (Fig. 107b). For frame booms a section of the strap made from channel bars, set perpendicularly (Fig. 107c) is applicable, and for small pile drivers straps made from single corners (Fig. 107d).

The heads of pile drivers. Underpulley trusses. Underpulley trusses sustain very considerable forces, which should be transmitted to the supports of the trusses, to the transverse beams of the pulley landing and to the boom of the pile driver by the shortest path.

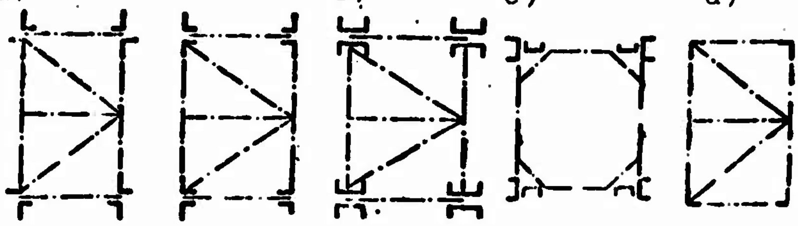

Fig. 107. Sections of simple booms of diagonal pile drivers.

The diagrams of the lattices of underpulley trusses are given in Fig. 108.

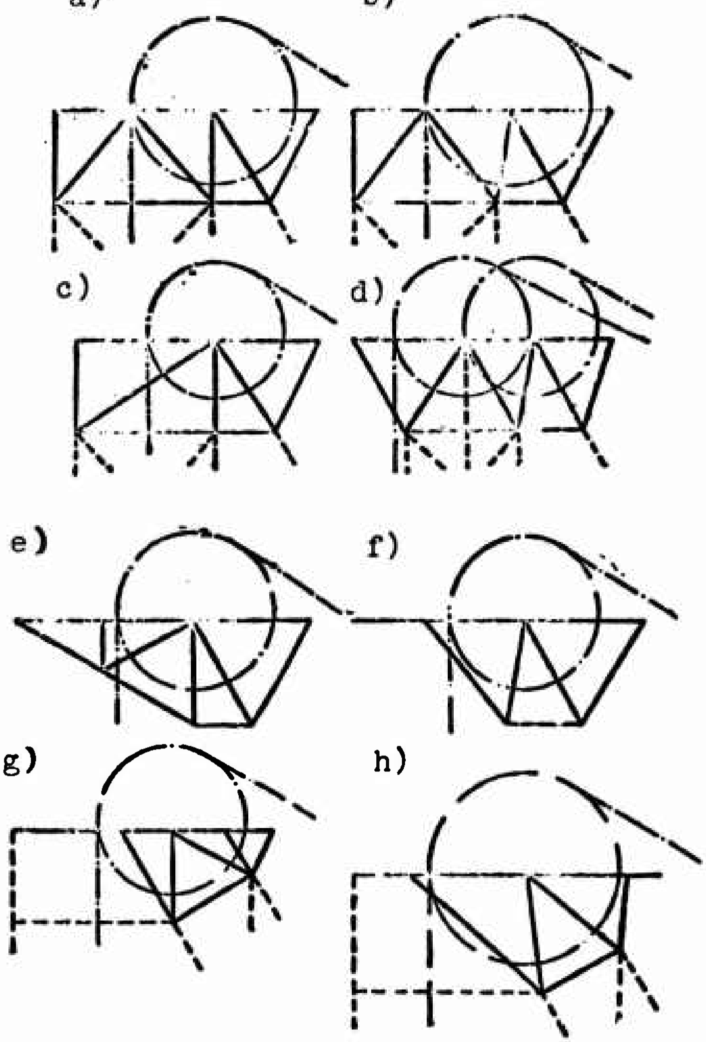

Fig. 108. The diagrams of the lattices of underpulley trusses: a), b), c), d), e), f) trusses of pile driver machine; g) and – trusses of diagonal pile drivers.

Figures 109 and 110 shows very simple underpulley trusses of the pile driver machines.

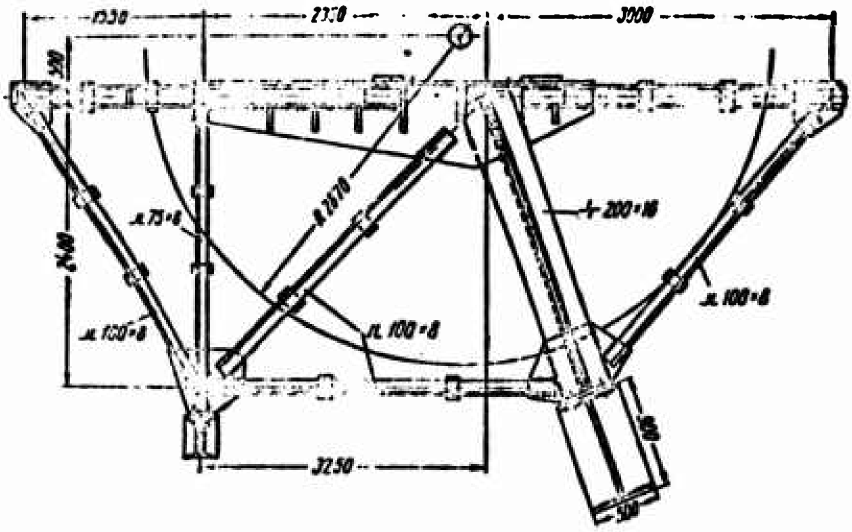

Fig. 109. Underpulley truss of a mine pile driver machine, which support pulleys 5 m in diameter.

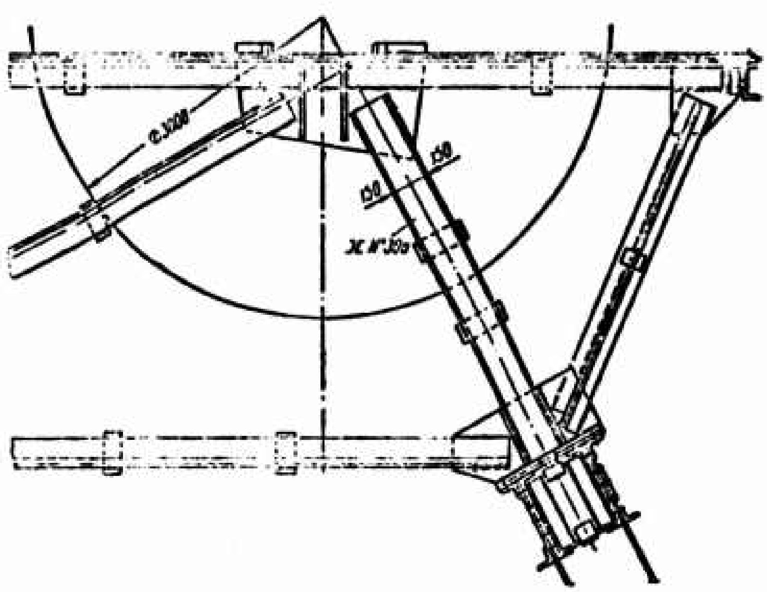

Fig. 110. Underpulley truss of a mine pile driver machine with pulleys 3 m in diameter.

The stability of underpulley trusses is assured by the transverse connections in the planes of the outer uprights, also installed in a number of cases in the planes of the intermediate elements of the lattice of the trusses. The transverse connections are more frequently arranged in vertical planes, and in a number of cases, also in inclined planes.

Horizontal lattice connections are usually installed at a level of the lower and upper straps of the trusses. In the plane of the upper straps of trusses in most cases, a rigid diaphram is created composed of straps of trusses, with auxiliary rods and solid metallic sheet flooring, spread within the confines of the entire landing, with the exception of breaks for the pulley devices. Horizontal connections usually extend to the planes of the main trusses or frames of the pile driver and are fastened to the elements of the latter, located at a level of the upper and lower straps of underpulley trusses.

The vertical transverse connections can be combined with the designs of transverse trusses. In this instance the elements of the transverse trusses are simultaneously vertical connections. Figure 111 gives a presentation of the simple transverse trusses.

With double-deck pulleys, the transverse trusses in the plane of the boom and in the plane of the machine are more complex. In connection with the intersection of the booms of an overall size of the lower pulley by a plane in the design, of the boom one usually introduces a portal, formed by the rigid frames (Fig. 111c, e) or by a truss (Fig. 111d). The first solution is predominantly used at moderate loads. The method in Fig. 111e, is expedient at limited sizes. The utilization of the trusses (Fig. 111d) is the most extensively used solution, which insures the least expenditure of metal.

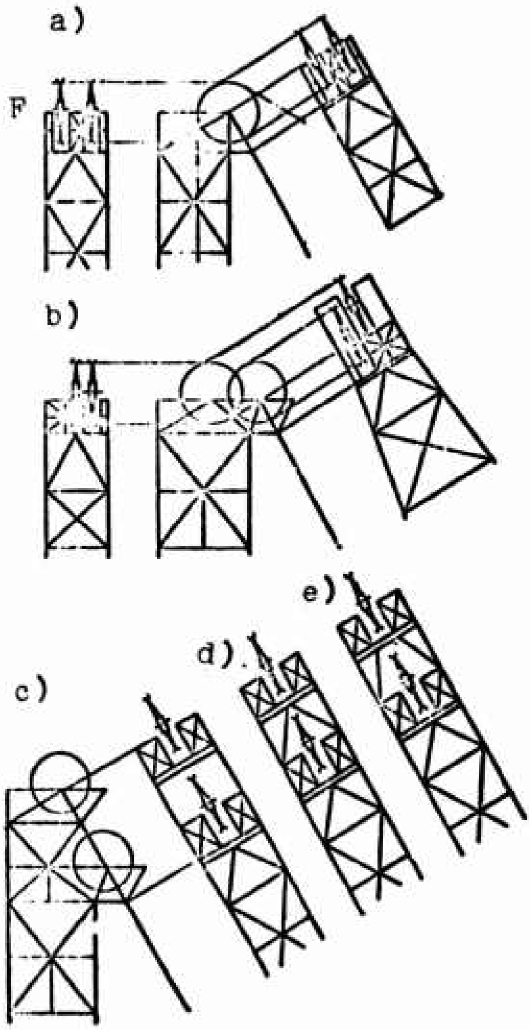

Fig. 111. The diagrams of transverse trusses of an underpulley landing: a) b) with the location of the pulleys at one level; c), e) with the location of the pulleys in one vertical plane; the portals are formed by rigid frames; d) with the location of the pulleys in one vertical plane; the portal is formed by the truss.

The bearings of the pulleys are centrally located relative to the underpulley trusses; therefore, the distance between the underpulley trusses are equal to the distance between the centers of the bearings. If several pulleys are placed on the underpulley landing then the following diagrams of positioning the underpulley trusses are possible.

Thus, Figure 112a, illustrates a diagram of the location (by design) of underpulley trusses using a device of two parallel pulleys on the landing, the axes of which lie on one straight line perpendicular to the plane of the lift. For a device using two pulleys based on the specified diagram it is necessary to have four

underpulley trusses, which support the loads from pulleys, transmittable through the bearings. Each of the trusses takes, in this instance, a load from one bearing, and the resultant force in the lift rope is distributed to two underpulley trusses.

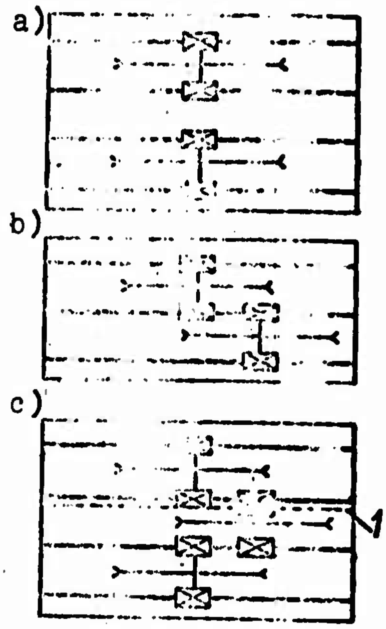

Fig. 112. Some schemes of underpulley landings: a) two parallel pulleys, the axes of which lies on one of the lines, perpendicular to the plane of lift; b) using two bearings of various pulleys on a central underpulley truss; c) a variant of the location of the underpulley trusses; 1 - truss with spread chords (subvariant).

With a certain movement of one pulley along the horizontal in the plane of the lift (Fig. 112b,) the number of underpulley trusses can be reduced by means of using the central underpulley truss for the device having two bearings of various pulleys.

In Fig. 112c, one of the three pulleys is moved in its plane lengthwise along the landing, because one of the underpulley trusses has been shortened. In this instance, just as according to diagram b, the distance between the axes of the underpulley trusses is equal to a distance between the pulleys.

Sometimes based on the conditions of the location of lift vessels, the diagrams, presented in Fig. 112c, require an excessive spacing of the middle trusses. In this instance the utilization of

the diagram, shown in Fig. 112c, is possible but the distance between the bearings of the pulleys and between the axes of the middle trusses in this instance are not equal; then one designs the central truss with the spread of the elements of the chord and gratings which however, results in the emergence of torsion of the truss chord and so on.

The designs of the underpulley landing should resist the action of a dynamic load suitably, possess a high degree of rigidity and stability. The structure, to which the pulleys are installed, should prevent the rapid run of the pulleys from alignment and should be of identical rigidity. Therefore, the calculations of the sags of the trusses with the further thorough checking of the possible displacements of the axes of the pulleys in the bearings in other cases should be performed. As a rule, one ought to avoid using such schemes.

The solutions of the underpulley landings are modified depending on a system, the material and the design of the pile driver, location and the sizes of the pulleys. A number of other features of a lift and of applied schemes also, to a considerable degree, can have a bearing on a solution of underpulley landings.

In the given examples underpulley landings are described, based exclusively upon the utilization of pivotal underpulley trusses. This solution is basic and always expedient in view of the large concentrated loads and their fully definite and constant position in space. In this way, the external loads can always be concentrated in the joints of the underpulley trusses and transmitted to the supports of the latter by the shortest way.

In practice one finds cases of using underpulley beams of a continuous section, which are widely used at underpulley landings with wooden structures and with ferroconcrete underpulley landings. The metallic underpulley beams of a continuous section are used in

the metallic frame pile drivers, which possess a small number of rods with developed cross pieces of continous section, uprights of frames and underpulley beams.

The beams of continuous section are usually also used in sinking pile drivers.

Heads of large pile driver machines and special solutions of underpulley landings. Described above were the solutions of underpulley landings for the pulleys of approximately identical diameter. In a number of cases combinations of the pulleys of various diameters are possible. So, for example, one of two lifts can be equipped with heavy pulleys, differing sharply in the sizes from the two other pulleys.

In this instance the resultant forces in the cables of the large pulleys considerably exceeds the magnitude resultant forces of the cables of the small pulleys and is passed outside the boom (Fig. 113a), which results in the emergence of considerable forces in the elements of a head and in the pile driver machine. Furthermore, the underpulley trusses are relatively complex, the load from the large pulleys is applied in the least profitable way, and the paths of the main forces are not uniflow to a sufficient degree.

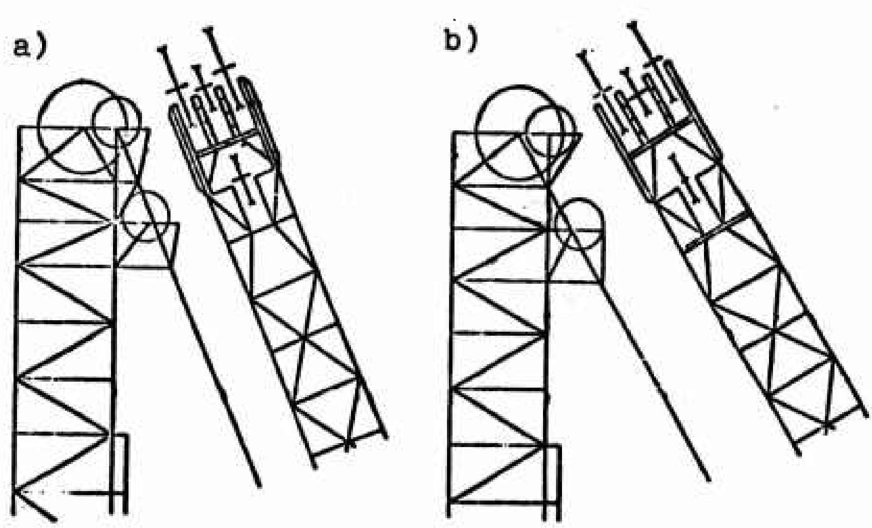

Fig. 113. The schemes of the head of a pile driver during lift, equipped with pulleys of various diameters: a) the boom is directed towards the outer pulleys; b) the boom is directed towards the large pulleys.

With the direction of the boom into the center of the large pulley, the head of the pile driver acquires a more rational profile (Fig. 113b). In this instance the direction of the boom coincides with the direction of the greatest resultant forces of cables, because of the considerably reduced forces in the elements of the trusses of the head. In the elements of the pile driver machine with this solution considerably less forces appear, as a result of which the weight of a pile driver is somewhat shortened.

An example of the head of the pile driver with different pulleys is given here not for the proof of the special advantages of scheme b, but for the distribution of conclusions to the solutions of the head of many pile driver machines. One ought only to have in mind that in a number of cases special combinations of pulleys are possible, especially, with various diameters. In other such and particular cases as well it is frequently expedient to introduce definite changes in the usual schemes of the head of pile drivers.

Single-lift and double-lift pile driver machines of considerable height, which support pulleys of even the largest diameters usually differ little from the above described pile driver machines of less height, which support pulleys of average sizes. It goes without saying that in the selection of the sections of elements of the trusses of those or any other pile drivers the magnitude of forces are considered, in accordance with what sections of the elements are assigned, and what single or double branches of the grating of the trusses and so on, are taken. In this case, however, the schemes of underpulley landings can remain practically constant because their selection depends mainly upon the number of pulleys and their location in space.

With a substantial increase in the number of pulleys of the head of pile driver machines specific outlines are taken. The heads of large mine pile drivers support up to ten pulleys having a diameter of 5-6 m. The height of such a head is approximately equal to

$$(D+1)n,$$

where D — the diameter of the pulley, m; n — the number of tiers of underpulley landing.

Thus, the height of a head with pulleys having a diameter of 5-6 m constitutes: with 3 tiers – up to 18-20 m, with 4 – up to 24-28 m, with 5 – up to 30-35 m, with 6 – up to 35-40 m.

The largest spans of underpulley trusses of the heat attain 15 m with the length of the trusses up to 20 m.

The described heads of the pile drivers are heavily loaded construction. Based on the character of the grating of the trusses and on the general overall sizes of the head, the latter can be properly compared with triangular-hipped pile drivers, the solution of which is substantially complicated by the presence of numerous joints, underpulley and transverse trusses, penetrating the head at various levels. If we take into account that the spans of a number of underpulley trusses are considerable, then with the calculation of the heavy loads of this part of the pile driver, the expenditure of metal, necessary for the manufacture of the head of a pile driver and the entire construction, will be very high. The weight of the large pile driver machines with multiple-level heads attaines 300-400 t.

Given below is a brief description of a four-level head of a pile driver machine, supporting 5-meter guide pulleys. Figure 114 shows the main truss of a head and the front truss, and in Fig. 115 – a boom frame and a diagram of the underpulley trusses on all four tiers of the head of the pile driver. Figure 114 shows the main truss of a head and the front truss, and in Fig. 115 – a boom frame and a diagram of the underpulley trusses on all four tiers of the head of the piledrivers.

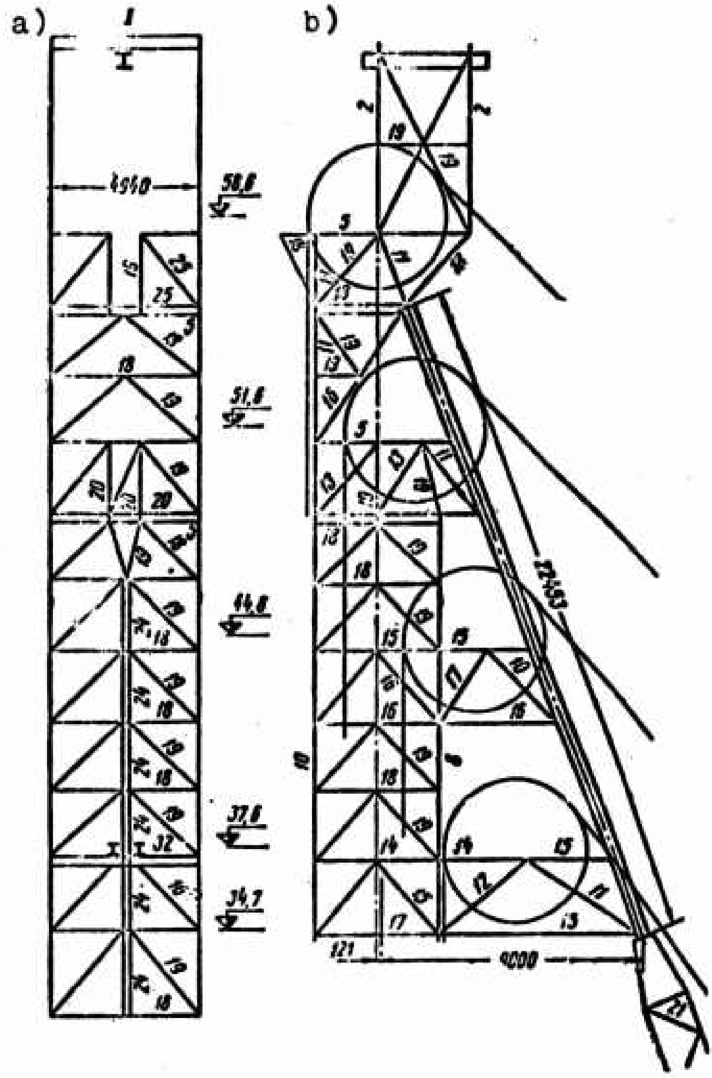

Fig. 114. The head of a large pile driver machine: a) front truss; b) main truss.

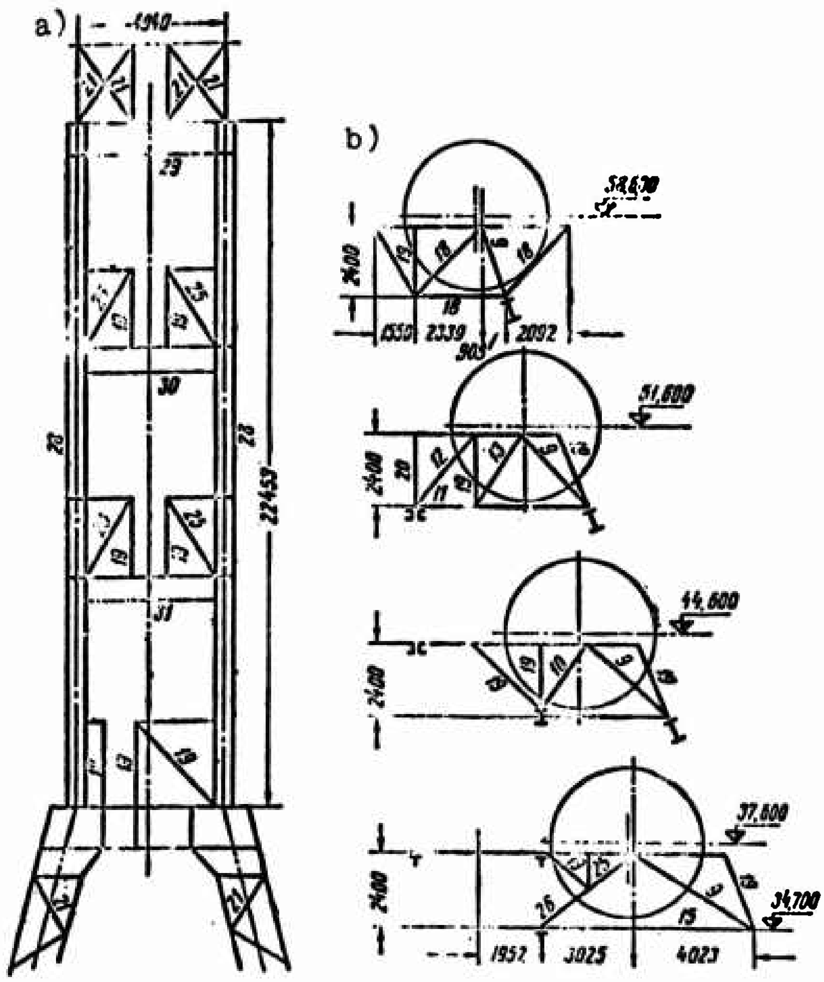

Fig. 115. The boom frame (jib) of the head of a large pile driver machine; a) boom frame; b) schemes of the underpulley trusses, set at various levels of the head.

The sections of the rods 1, 2, 3 are presented as No. 45 double-T; 30; 20; rods 4 and 5 – paired channel bars No. 20 and 16; the section of a rod 6 – by channel bar No. 16, and rod 7 – by two channel bars, No. 14. Rods 8-10 are composed of paired angles 200 × 18; 200 × 16 and 150 × 12; rods 11-20 – paired angles 120 × 10; 100 × 10; 100 × × 8; 150 × 16; 150 × 12; 120 × 10; 100 × 10; 100 × 8; 75 × 8 and 75 × × 8; rods 21-24 – angles 100 × 8; 120 × 10; 75 × 8 and 65 × 6; rods 25-27 – paired angles 65 × 6; 150 × 16 and 100 × 75 × 8. Finally, rods 28-31 (welded double-T sections) with straps and walls made from sheets 500 × 20 and 660 × 12; 500 × 30 and 840 × 20; 500 × 20 and 760 × 20; 500 × 16 and 760 × 16.

The heads of diagonal pile drivers. The equipment of the head of a diagonal pile driver is shown in Figs. 116 and 117. The underpulley truss of the diagonal pile driver in this instance is characterized by a small span and by the central application of the load; the supporting reactions of the truss are close based on value. Horizontal an inclined junction plates are furnished on the supporting sheets of the truss for bracing in the joint of the connections on the lower chords of the underpulley trusses and inclined connections. A horizontal sheet is provided on the upper chord of the truss and supporting planks for the bracing of the bearing.

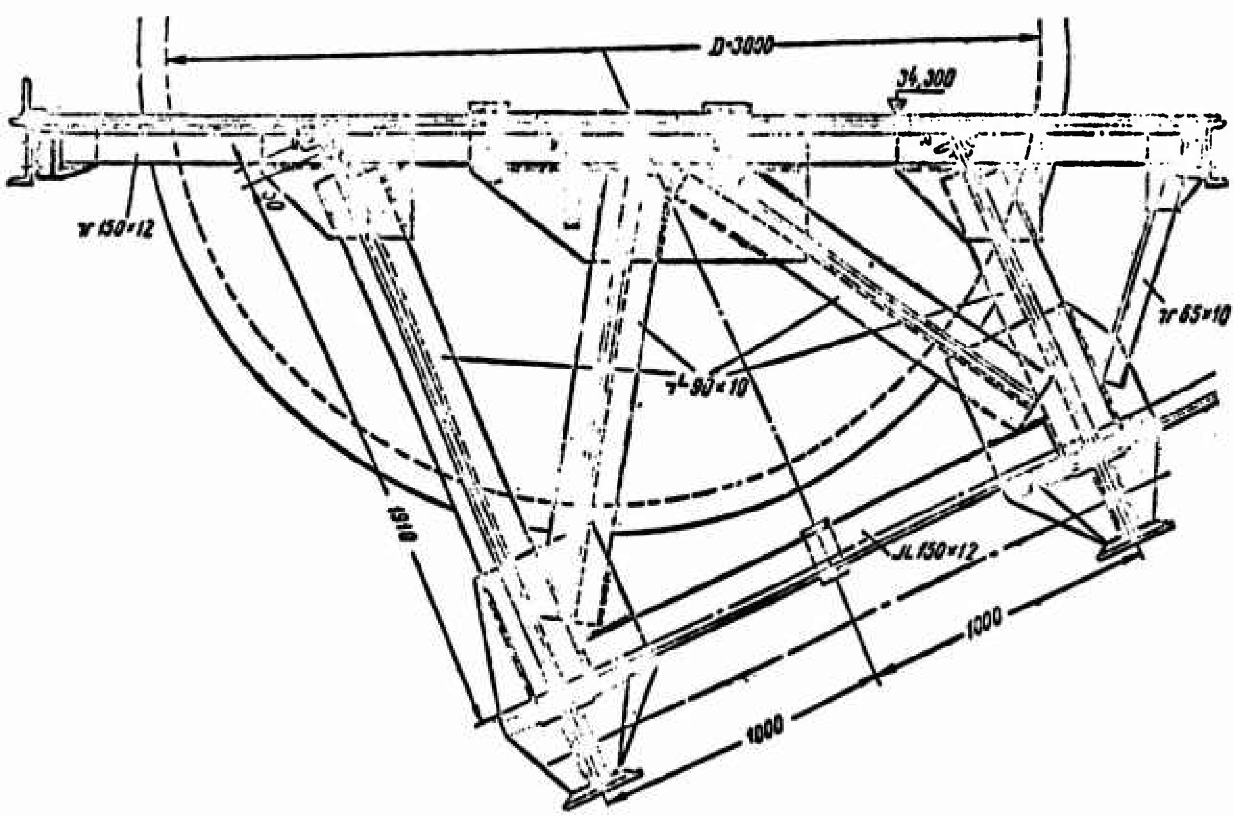

Fig. 116. Underpulley truss of a diagonal pile driver.

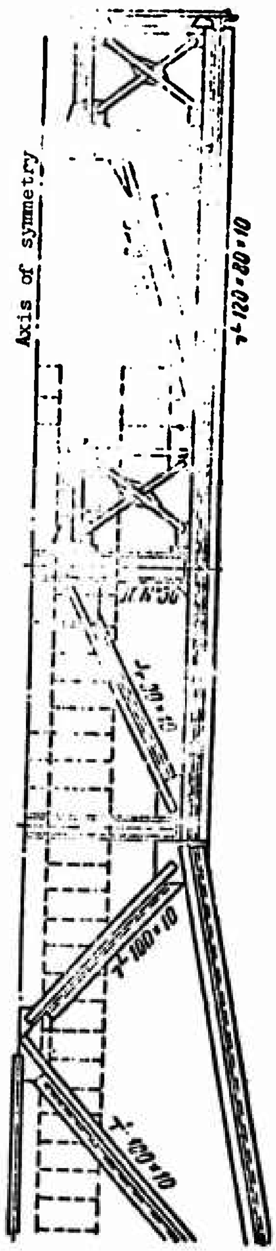

Fig. 117. The transverse frame of the head of the diagonal pile driver.

The solution of the transverse trusses of the head of the diagonal pile drivers is similar to the earlier described designs. The underpulley trusses are installed to the transverse truss or portal frame (Fig. 117), of the overlapping pulley on the lower underpulley landing. Underpulley trusses both when using the pulleys on one or two or several underpulley landings are analogous, have identical spans, sizes and sections and can easily be unified. The magnitude of the spans of underpulley trusses is small and it corresponds to the reach of the branches of the boom.

One should strive for the supporting reactions of every underpulley truss to be as close as possible to the magnitude, which will approximately guarantee the equal forces of compression of the branches of the boom and the uniform sections of the straps of the latter.

3. Metallic Pile Drivers

Figure 118 shows a diagram of standard skip pile drivers having heights of 34 and 46 m, used in the coal industry. The levels of the receiving funnels of these pile drivers are equal to 20 and 32 m. The pile drivers - single-hoisting, on an underpully landing are installed at one level for two pulleys 4 or 5 m in diameter. The section of the pile driver machine in a plane in the axes of the uprights is equal to 3.2 × 4.5 or 3.2 × 4.7 m. The height of the panels of the machine is 3 m. The section of the uprights of a machine - a cross made from the two corners 120 × 120 mm. The section of the boom - welded double-T 600 mm in height, the width of belts, 350 mm. Weight indexes are presented by curves 3 and 4 in Fig. 70.

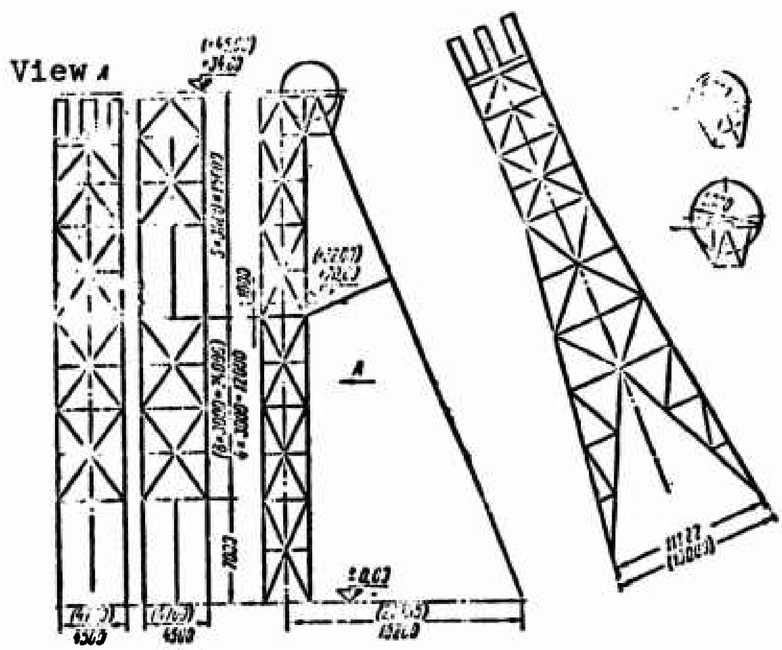

Fig. 118. The diagram of the standard single-hoisting coal shaft skip pile drivers 34 and 46 m in height.

Figure 119 shows the diagram of the standard coal shaft cage pile driver. The pile driver carries two guide pulleys of one double cage lift. The lift vessels - overturning cages.

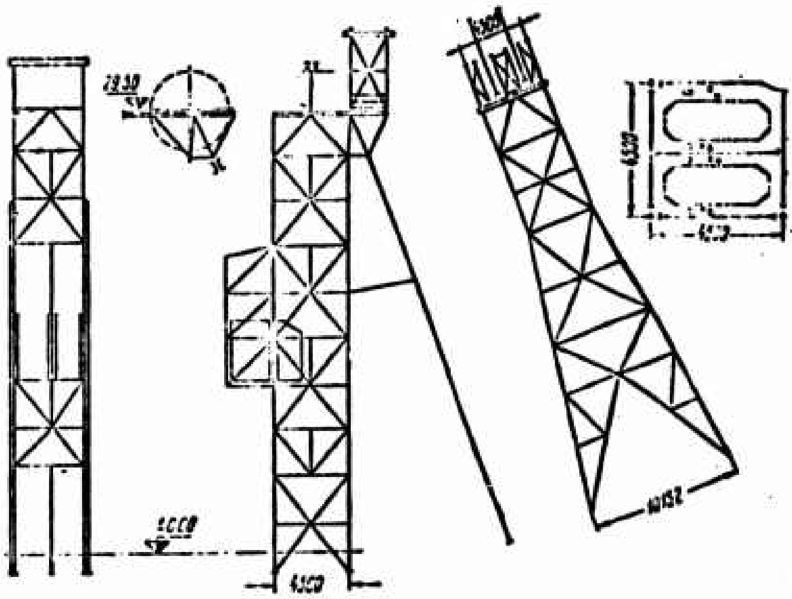

Fig. 119. The diagram of the standard coal shaft single-hoisting cage pile driver of 29.5 m in altitude.

The diagram of the mine skip-cage pile driver machine is given in Fig. 120. Upper pulleys are located at a height of 67.2 m above the level of the collar of the shaft. A pile driver carries six guide pulleys 5 m in diameter. The pulleys of the upper landing sustain the loads of the two-skip lift, the remaining pulleys carry the loads of two lifts from two doubledeck cages and two counterbalances. The breaking force of the skip rope of this pile driver is equal to 279 t, the cage rope - 194 t, the working forces are equal to 31 and 19 t respectively. The pile driver is equipped with cable parachutes [PTK] (ПТК). The load with the braking cage constitutes about 80 t. The weight of the given pile driver without planking is equal to 313 t, which can be explained by the presence of three lifts, by the great height of the pile driver and by the increased area of the section of the machine by design.

Figure 121 shows a cage double-lift mine pile driver 58.6 m in height; the diameter of the pulleys is 5 m, the diameter of the lift

ropes is 56.5 mm; the breaking force is 220.5 t. The weight of the pile driver constitutes 200 t, which can also be explained by the increased section of a machine by design, by the presence of two lifts and by the considerable height of the pile driver.

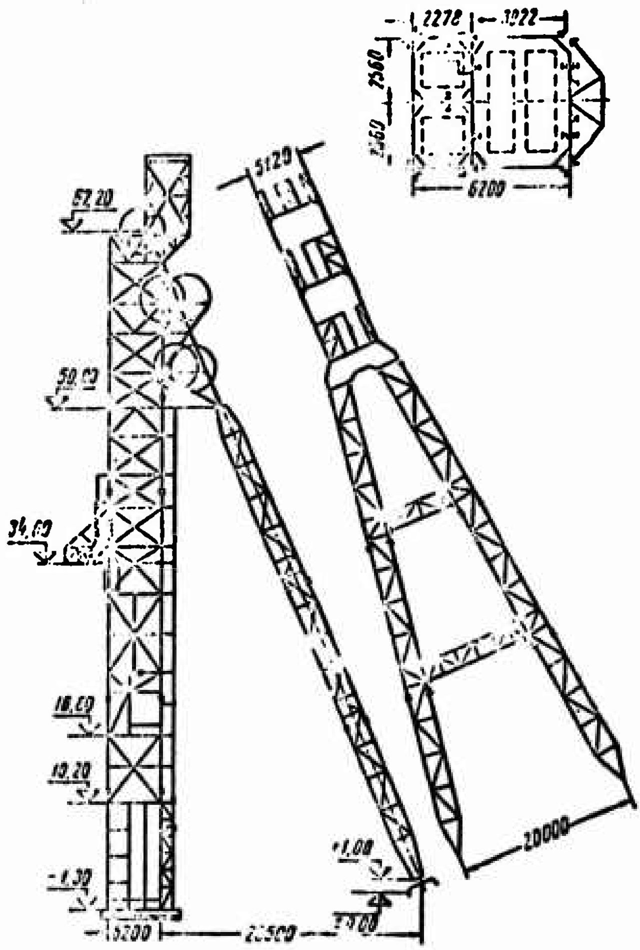

Fig. 120. The diagram of a large mine skip-cage pile driver machine.

The effect of the various causes on the increase in the weight of the single lift pile driver is tentatively evaluated by the following values (% for the data in Table 12):

two lifts (on Fig. 121) 20
three lifts (on Fig. 120) 30

loaded supporting frame with
increased spans (on Fig. 120) 10
height within the limits of 40-60 m
(on Fig. 121) 10
height within the limits of 60-80 m
(on Fig. 120) 20
the increase in the section of the
machine (according to Fig. 120/according
to Fig. 121) 20/10

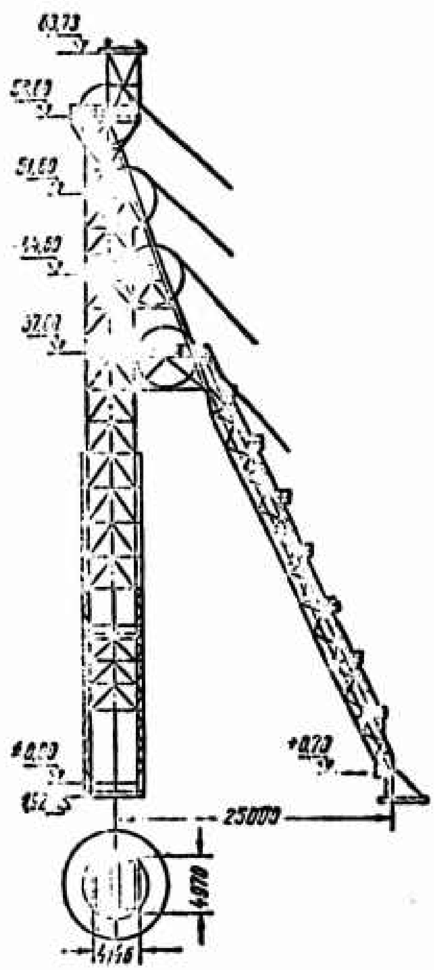

Fig. 121. The diagram of the large cage double-lift mine pile driver.

The diagonal metallic pile driver (Fig. 122) is constructed in such a way that the loaded boom with almost centrally located sizeable forces can be represented by a simple symmetric spatial design, formed by two pairs of uniform trusses. The pile driver mount, receiving comparatively small loads, has a frame design made from channel bars. A similar combination of trusses and frames for diagonal pile drivers is, apparently, expedient.

The underpully trusses of the lower underpulley landing area completely analogous to the upper underpully trusses presented in Fig. 116. In both cases the underpully trusses have the identical spans, equal to 2 m, identical sizes and identical sections of the elements of the trusses; the uprights and bracings of the underpulley trusses are made using two cross-shaped set corners 90 × 90 × 10; the strap of the truss - from matching corners, 150 × 150 × 12.

The weight of the metal structure of the pile driver constitutes 62 t, and without the pusher and a part of the supporting frame, 59 t. Based on the data, presented in Chapter V, it is possible to determine the weight without the pusher and a part of the supporting frame of the conventional single lift pile driver machine, which will be equal to

$$aH\sqrt{P_{\text{ж}}}\cdot 1{,}1 = a\cdot 34{,}5\sqrt{57{,}2}\cdot 1{,}1 = 0{,}23\cdot 34{,}5\cdot 7{,}55\cdot 1{,}1 = 66 \text{ t}.$$

In this case coefficient a according to the graph given in Fig. 70, is taken to be equal to 0.23; coefficient, equal to 1.1, is considered to be the arrangement of the underpulley landing at two levels. Conversely, based on the weight of the pile driver, it is possible to set the actual value of the coefficient a. The latter is equal, in this case, to 0.205. One ought to note that the weight of the free butons are included as a value of the actual weight of the pile driver. The weight of the pile driver without the weight of a part of the free buntons, which extends beyond the point of the weight of conventional buntons, is equal to 56.5 t, which corresponds to the value of a coefficient a, equal to 0.195. The given indexes of the weight are 5-10% less that the corresponding indexes of the weight for the pile driver machines.

Pile drivers of the described type with a comparatively heavily loaded mount are also made with grated mounts. The diagonal

pile d iver of such a design, support pulleys 4 m in diameter and rope breaking devices, are given in Fig. 126. The pile driver is characterized by a large span of the boom, which is determined by an arrangement of tunnels and by other conditions as well. All planes of the given pile driver are presented by farms with conventional frame panels within the limits of the machine.

Figure 127 shows a diagram of a mine diagonal metallic pile driver 46 m in height, supporting different pulleys with the diameter of the ropes of the main lift, 60 mm, and the breaking force of the latter, 219.2 t. The pile driver differs by a large span of the boom at its base crosswise, consisting of 26 m, which can be determined by the specified position of the machine construction. A boom in this case is represented by two spatial frameworks, each transverse section of which is inscribed in a square. In this way, a pile driver is composed of two spatial frameworks and a frame-lattice design of the machine. The described solution can be considered as one of the most expedient among the high pile drivers, and under the given conditions - the most unified possible.

Pile drivers with two booms. As a variation of pile driver machines there are pile drivers with two braces, arranged in a plane at an angle of 90°. Also is an arrangement of the booms at an angle of 180° possible.

The presence of several booms approaches the design of delta-shaped pile drivers to that of polygonal, and as a rule, results in an increase in the expense of the construction material. Furthermore, the cost of two separate machine housings, which correspond to thermoficated, water-conducting, air-conducting and, energy pipes and ducts, couplings, additional passages and landings substantially exceeds the corresponding expenses using one unitized machine housing. In connection with this one ought to avoid using pile drivers with two booms. The second boom can be introduced only as an exception during reconstruction.

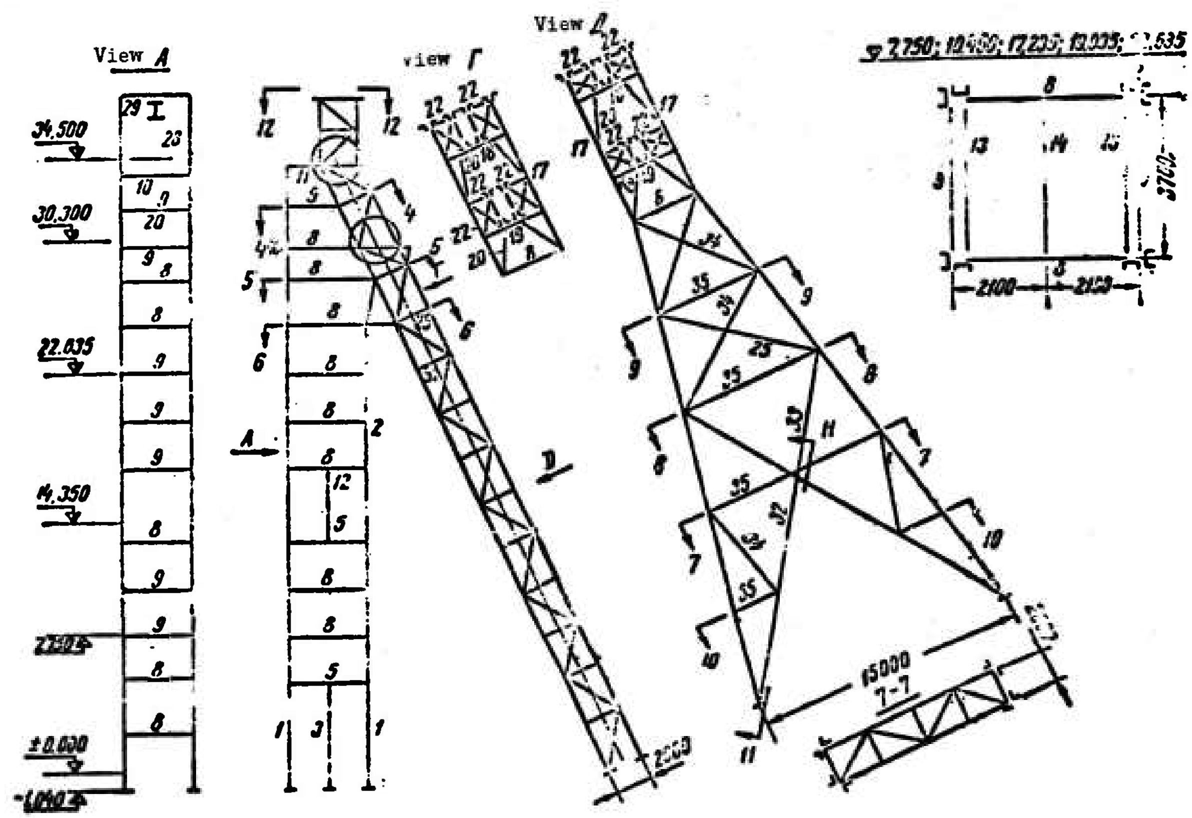

Fig. 122. The diagram of a diagonal metallic pile driver.

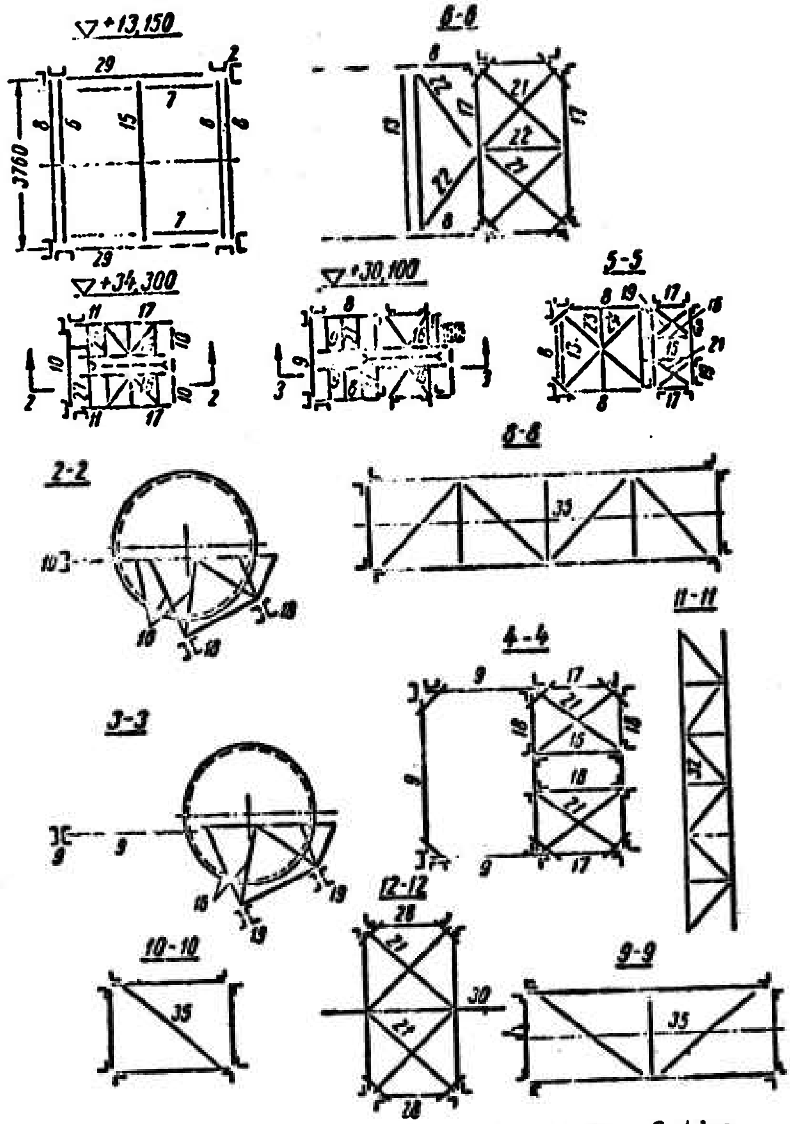

Fig. 123. Installation diagram of the underpully and unloading landing and the associated diagonal pile driver.

The pile driver carried two pulleys 3 m in diameter, arranged in one vertical plane at heights of 34.5 and 30.3 m above the level of the collar. The breaking force of the rope is 57.3 t.

The calculation of the pile driver is done by means of the parallel utilization of the method of forces and of the method of distribution of unbalanced moments.

Figure 123 (sections 2-2, 3-3, 4-4, 5-5, 6-6 and others) shows an installation diagram, elucidating the underpully and unloading landing equipment also in connection with the pile driver, and in Table 27 - a section of the installation brands. Figures 116, 117 illustrate the head of this pile driver; Fig. 124 shows a section of the pile driver machine, in which free buntons are used, connected wit the supporting designs of the pile driver; the mounts for these bunton can be made on the spot, simultaneously with the work on the reinforcement of the machine. Visible in this figure are the designs of uprights, crossbars and buntons. The main frame of the described diagonal pile driver is presented in Fig. 125.

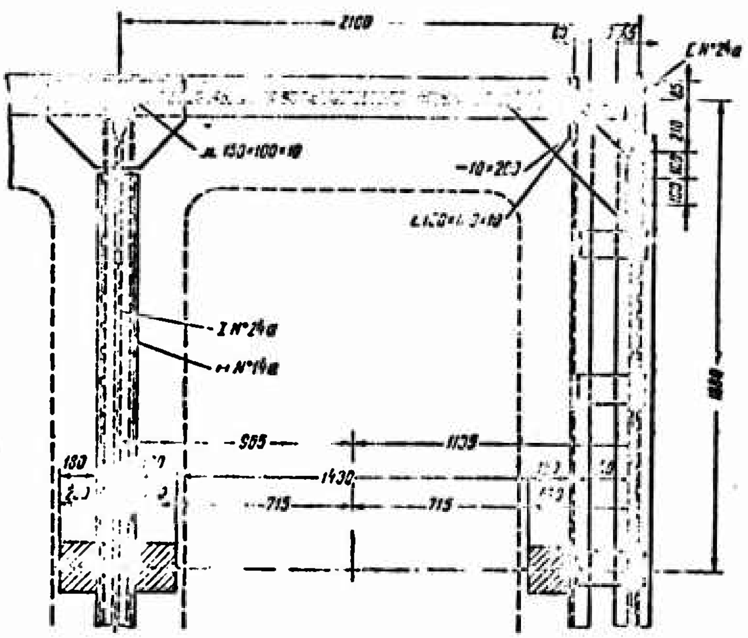

Fig. 24. The section of the pile driver machine using free buntons.

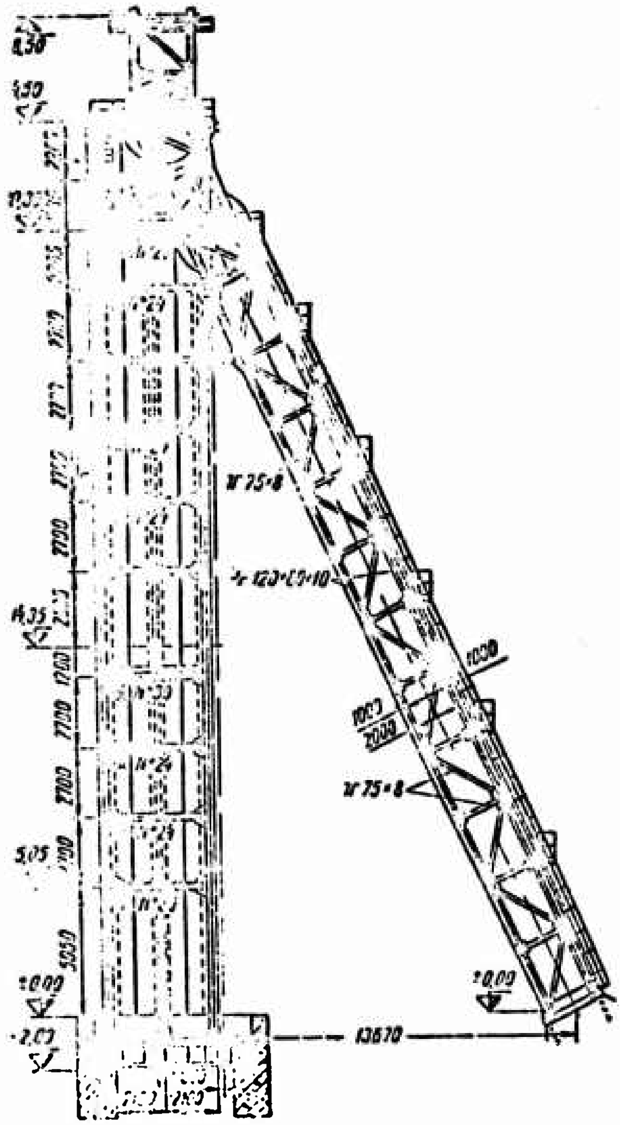

Fig. 125. The main frame of the diagonal pile driver.

Table 27.

No. of element	Designation of the element	Type of section	Composition of section
1	Uprights of the machine	][	[N°24a ⅃ 100×10
2	Uprights of the machine	]⌐	[N°24a
3	Uprights of the machine	┐└	∟ 100×10
4	Beams of the supporting frame	I	— 20×220 — 10×[illegible]
5	Beams of the supporting frame	][	[N°30a ∟ 100×10
6	Buntons	]	[N°30a
7	Underside stop beam	]	—10×440 [N°30a
8	Crosspieces of the machine mount	][	[N°24a
9	Crosspiece of the machine mount	][	[N°20a
10	Crosspiece of the machine mount	]	[N°20a
11	Crosspiece of the machine mount	┐┌	┌ 100×12
12	Beams of the pusher	⊔ [	⊔ N°20a [N°24a
13	Buntons	[	[N°24a
14	Buntons	⊔ I	⊔ N°14a I N°24a
15	Buntons	⊔ I	⊔ N°14a I N°30a
16	Underpulley truss	Complex	⅃∟ 150×12 ┐∟ 90×10
17	Trusser of the bracing of the pile driver head	Complex	┐∟ 120×80×10][N°30a][N°24a ⊢ 75×8
18	Transverse supporting beam	][	—10×220 [N°30a —10×220
19	Transverse supporting beam	][	—10×220 [N°30a

Table 27 cont'd.

No. of element	Designation of the element	Type of section	Composition of section
20	Supporting bracings	┘┌	L 5[illegible]×10
21	Couplings	┌	┌ 75×8
22	Couplings	┐┌	┌ 75×8
23	Buntons	I	I N°24a
24	Horizontal couplings	L	L 1[illegible]0×10
25	Combined bracing	[illegible]	L 75×8
26	Beams	[	C N°14a
27	Flooring of the landing	[illegible]	−10 ┌ 65×8
28	Mounting frames	Complex	┐L 12[illegible]×63×10 ┐┌ 75×8
29	Cross pieces	][	C N°30a
30	Monorail	I	I N°30a
31	Truss of a branch of the brace	Complex	┐L 120×80×10 ┐┌ 75×8
32	Trusses of the lattice of the brace	Complex	┐L [illegible]20×80×10 ┐┌ 75×8
33	Lattice of the brace	┐L	L 150×12
34	Lattice of the brace	┐L	L 100×10
35	Truss of the spacer of the brace	Complex	┐┌ 120×80×10 ┐┌ 75×8
36	Vertical stairs	┼−−┼	L 75×8 Ø 18
37	Stairs of the brace	☰	Ø18
38	Element for the bracing of the rails	L	L 120×80×10

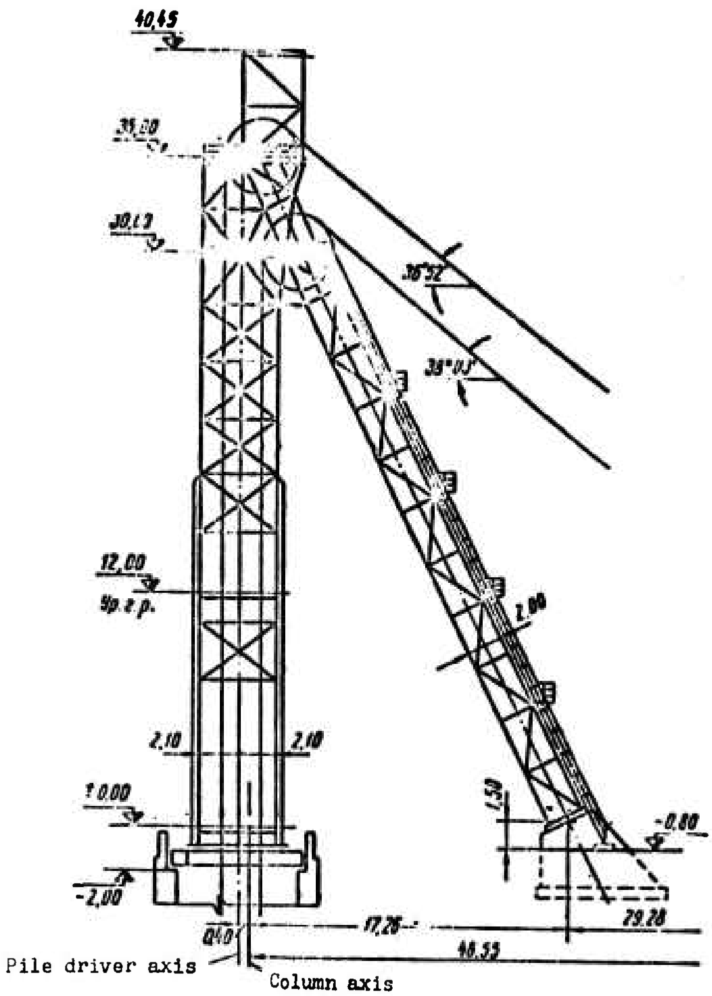

Fig. 126. Diagonal pile driver with trusses in the main and transverse planes.

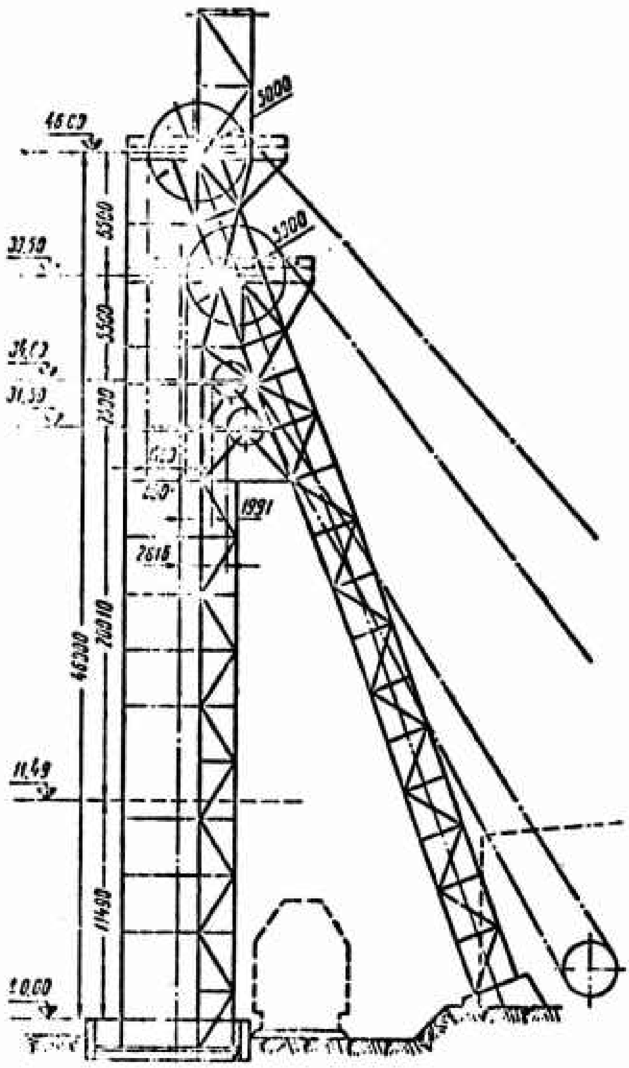

Fig. 127. The diagram of a diagonal metallic pile driver with height of 46 m with a support boom, equal to 26 m, and with a machine of frame-lattice design.

Designs of frame pile drivers. The designs of diagonal pile driver machine and pile drivers of other systems can be made from frames.

Above were given examples of a diagonal metallic frame pile driver (see Figs. 122 and 125). The pile driver in Fig. 125 with a height of 34.5 m and a breaking force of the rope of 57.2 t weighs 56.5 t, and is characterized by a value of the coefficient a in the formula of the weight of the construction equal to 0.195, which is lower than the normal values. A metallic frame pile driver machine with a height of 25 m and with a breaking force of the rope of 57.2 t weighs 40 t and is characterized by a value of the coefficient a in the same formula of the weight of construction equal to 0.21 which is also lower than the normal value of a.

The diagram of one of the largest frame pile driver machine is given in Fig. 128. In this pile driver the frame designs are provided with a head, boom and a machine mount. The design of the latter with a section, 6376 × 7276 mm, is represented by continuous sheets 8 mm in thickness reinforced in the angles of the mine shaft with corners, 150 × 150 × 12, and 150 × 100 × 12, but in the planes of the walls of the machine - by vertical ribs spaced at a distance of 1.05-1.35 m and by horizontal No. 30 channel bars spaced at a distance of 2.2-2.4 m based on the height of the machine.

The uprights of boom have walls of variable section made from sheets 14 mm thick having a height of 1452 mm towards the bottom, and 2652 mm towards the top, the belts - made from sheets, 24 × 480 mm. The transverse span of the boom is equal to 18 m; in this plane a double semi-bracing lattice made from channel bars, is provided. The head of the pile driver is formed by frames. The uprights of the inclined boom of the frame within the limits of the brand are 42.0-52.0 m, and are composed of sheets, 1860 × 12, and 2 × 400 × × 20 mm.

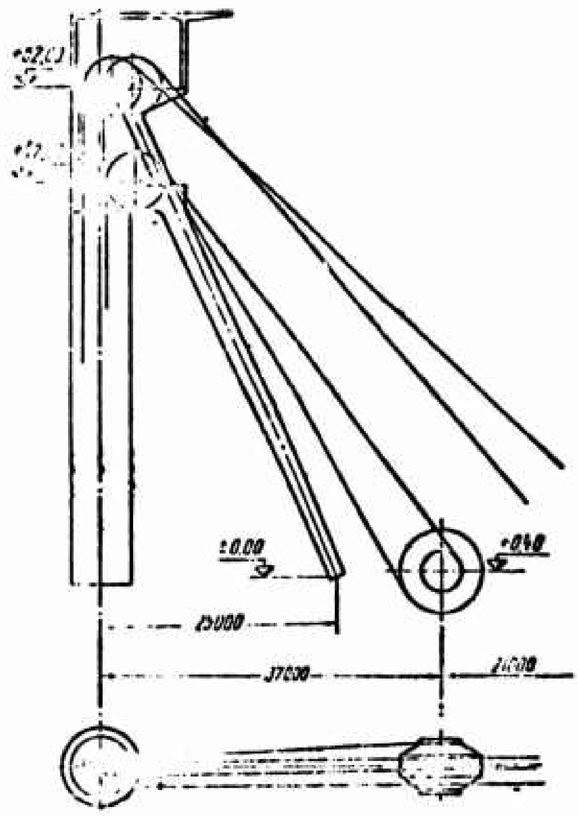

Fig. 128. The diagram of the frame metallic pile driver machine.

The described frame pile driver is double-hoisting, carries pulleys 6 m in diameter using ropes 60.5 mm in diameter with a breaking force of 212.5 t. The weight indexes of this frame pile driver are not favorable. The weight of the steel designs of the construction is equal to 455 t; furthermore, the weight of the special anchor bracings constitutes 4.8 t, the weight of the rail guides and fastening parts is 17.3 t. One should, however, take into account the presence of a number of unfavorable circumstances, causing an increase in weight (because of the large sections of the machine mount - by 35%; because of two lifts - by 15%; height - 10%; the increase in the spans of the supporting frame and the presence of special equipment - 20%, the whole - by approximately 80% in comparison with the weight of the normal single-hoisting pile driver). Excluding

the weight of the steel designs, the minimum weight of the hermetically sealed sheating (45 t), can produce a tentative value of weight coefficient *a* (in the formula of a weight with a root of the second degree), which resulted in the indexes of the single-hoisting pile drivers, is equal to 0.30.

Outside the dependences on the given indexes, one ought to make note of the applied partial solutions in the given pile driver.

As a result of the presence of weak ground the equipment which assures the lift, and in part, the lowering of the machine mount and boom, are provided. Specifically, a mounting can be moved on a vertical line using hydraulic jacks with a bearing capacity of 100-200 t. Furthermore, horizontal movement of the machine within the limits up to 90 mm is also provided using screw jacks or stands. For this purpose the underside pile driver beams have cantilevers 1, mansonry and an anchor of a specified design, composed of two parts lengthwise (Fig. 129).

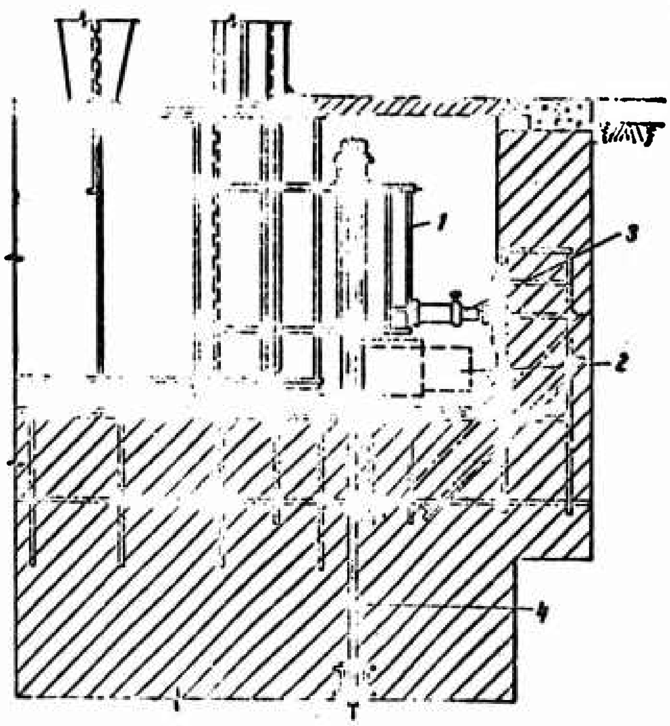

Fig. 129. Attachment of the underside pile driver beam, which allows movement of the machine: 1 - the cantilever of the underwide pile driver beam; 2 - jack; 3 - screw stop; 4 - components of the anchor bolts.

The majority of other parts is not characteristic just for this construction; therefore, it is possible to note only the following: in each tier of the crosspieces of the machine mount in the elements of the rigidity of the machine have been provided at all angles. Besides the hermetically sealed windlasses with doors for the entry of vessels and elongate material, handled from the main landing, and also from stairs, and clamps attached to the machine mount, a number of windows for the natural illumination of the machine and hermetically sealed doors on the landings for the servicing of the skips and unloading curves, has been provided. For a lift on pulley landings there are stairs. Furthermore, for the inspection and repair of interior equipment of the pile driver on the mount vertical stairs for the steeplejacks is provided. In this case durable handrails are set on every crosspiece of the machine (2.2-2.4 m by height) as an interior by-pass of the walls of the machine on crosspieces.

The valves of the machine for paying out the lift ropes can be installed and removed using suspended lift ropes; for setting up installation and repair scaffolds clamps are provided on the machine, for the alignment of the pile driver during installation and actual operation there are control apertures in the machine and in the boom, for unloading curve overturning skips, type P-38 rails are used. On the unloading levels dust-removing devices are set up. Lower down, the skip separation is made with a continuous partition made of sheet steel 4 mm thick. On the pile driver oil pan housings of the pulleys and an oil trap pipeline are provided; shockproof ropes, supported by guide tires and, the stop beams, calculated for breaking force, are employed. The lift equipment for the pulleys is made with special shaft cross-members for attachment to lift units; the lifting accommodated with the aid of a hand winch. All bolts, nuts, washers, screw stops, the upper parts of anchor bolts, spacing sheets under the machine have been galvanized.

Briefly described below are metallic pile drivers, which possess a number of characteristics also used in the copper-ore

industry. These pile drivers have a frame design with a small number of the crosspieces, adapted basically to underpulley, unloading and other landings (Fig. 130). Such a pile driver can be classified with the A-frame type.

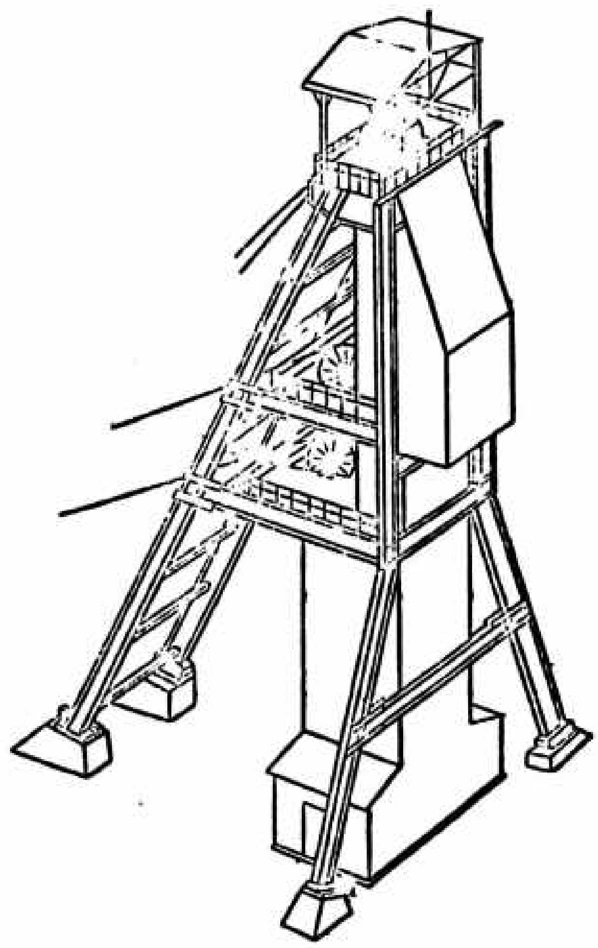

Fig. 130. A view of frame metallic half-hip pile driver.

The pile drivers and corresponding lift devices being considered are characterized by mutually associated solutions. The pile driver supports the pulleys from not more than two lift devices, and lift machines are arranged radially. The skip lift machine is located near the middle plane of the skips, at right angles to the latter or at a certain small angle to the normal. The cage lift

machine is located in the immediate area of a housing of the skip machine. An angle between the plane of the cage lift and the vertical plane, passing through the small axis of the cage (by design), goes up to 20°.

In accordance with the location of the lift machines the pulleys of the deckhead pile driver are installed. On the upper underpulley landing the pulleys of the skip lift are set at a small angle to the plane of the pile driver. On the lower underpulley landings the pulleys of the cage lift are set at a large angle to the same plane.

The front vertical frame of the pile driver is distributed on the base. The supports of this frame are located on the outside of the over-all dimensions of the shoring of the stem of the shaft. The support designs of the construction are not connected to the structures of the shoring of the stem of the shaft. The metallic mount is suspended on the supporting designs of the pile driver (Fig. 131). On the plane of the abutting to the shoring of the stem of the shaft, the mounting of the pile driver is not quite finished in front of the ferroconcrete designs of the shoring of the mouth. With the help the special rods made from pipes or other shapes, which are shifted relative to the vertical line of the design of machine to the shoring, is assured. For this, pipes 100 mm in diameter are inserted in the apertures of several horizontal elements of the machine and welded to matching parts of the shoring. In this way, the shoring of the stem can be shifted along a vertical line relative to the pile dirver and can sustain horizontal loads from the machine.

The upper and lower joints of the main frames of the described pile drivers are self-explanatory in Fig. 130; the intermediate joints are shown in Fig. 132.

Fig. 131. A view of the frame of a metallic half-hipped pile driver.

The weight of a single-hoisting frame metallic pile driver having a height of 32 with a breaking force of the rope of 57.2 t is equal to 91.8 t. The weight of a conventional metallic pile driver machine under analogous conditions is equal to 56.5 t. In another case, a double-hoisting frame pile driver of the described type of the same height with a breaking force of the rope of 69.3 t weighs 116.6 t. The weight of a corresponding pile driver machine constitutes 70 t. The weight indexes of the examined A-frame pile drivers, specified for the weight indexes of single-hoisting pile drivers, are characterized by a value of coefficient a (in the formula of a

weight with a root of the second degree), equal to 0.38, which characterizes the increase in the weight of steel structures in comparison with mounted and diagonal pile driver machines.

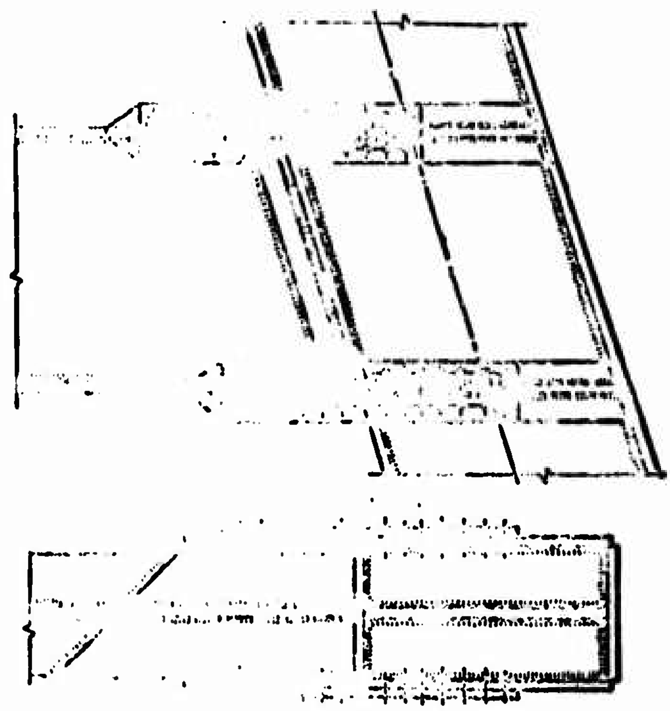

Fig. 132. The joints of the frame of a metallic half-hipped pile driver.

One ought to note that the increased weight indexes using radially arranged machines are characteristic not only for the described A-frame pile drivers, but are also characteristic for grated A-frame metallic pile drivers. According to the description of a group of deckhouse pile drivers, operating in Witwatersrand enterprises, the mean value of the coefficient a constitutes about 0.61. Taking into account that the pile drivers are of this double- and triple-hoisting group one can assume that the weight of the built-in hoppers and landings constitutes about 15% of the weight of the construction. In this instance with an overall decreasing coefficient of the weight of 0.7, the mean value of the weight of the coefficients of these grated pile drivers (in the formula with

a root of the second degree), specified for the weight of a single-hoisting pile driver, is equal to 0.42, which is higher than the indexes of the weight of the above described frame pile drivers, and which corresponds to the double weight of the corresponding mounted and diagonal pile drivers.

The presented examples confirm the given position in Chapter IV about the expediency of using frame pile drivers with an average and small height and with relatively moderate loads. With an increase, specifically, in loads and with their disadvantageous arrangement (which exists, for example, with a radial mounting of the lift machines), with an increase in the number of lifts up to two and three, and in other unfavorable cases, the utilization of frame pile drivers should be limited.

Pile drivers with depressed booms, boomless and guy-wire supported pile drivers. Based on the described systems it is possible to produce a large quantity of various pile drivers. These designs can differ from the basic one to such a degree that the obtained construction can pertain to other system. Thus, for instance, Fig. 133 gives various diagrams, obtained from the simplest mounted pile driver machine. The pile driver, presented in Fig. 133b, differs somewhat by a depressed boom which frequently is the case for mounted pile drivers with a developed head, especially, with a large number of pulleys (see Fig. 114). Sometimes only pile drivers of such a design can be used.

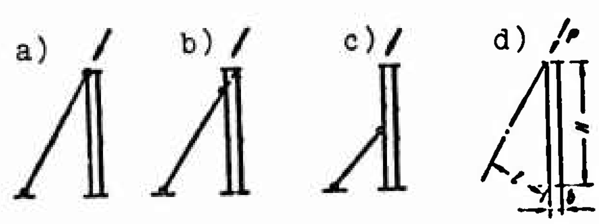

Fig. 133. Various diagrams derived from the simplest mounted pile driver machines with changes in the boom: a) mounted pile driver machine; b) mounted pile driver machine with a depressed strut; c) pile driver cantilever formed by the introduction of a semi-boom; d) boomless pile driver.

It was shown above that mounted pile drivers with somewhat depressed booms in certain cases are also expedient. Specifically, this is observed when using two lifts and with relatively large loads of one lift. In this case (See Fig. 113) a certain lowering of the boom along with its displacement and approach in a direction of the resultant forces of the main lift is entirely appropriate.

Certain metallic double-hoisting pile drivers about 40 m high are made with a depressed position of the booms. In actual practice, the operation of these unorthodox pile drivers were not observed. The structures were characterized by a high degree of rigidity. One of the pile drivers were subjected to the effect of specific loads as a result of the rupture of the lift rope whereas the structure did not suffer damage. The weight indexes of the pile drivers with limited depressed booms are analogous to the corresponding indexes of conventional mounted drivers.

A further depression in the boom leads to the fact that the mounted engine operates as a powerful cantilever-arm bracket (Fig. 133c), firmly embedded in the lower part, and at the point of abutting of the semi-boom. The diagram, presented in Fig. 133d, is characterized by a complete absence of a boom. In such a pile driver nothing remains of the basic system and the construction is a simple cantilever - *a boomless pile driver* (the most widely used type of cantilever pile drivers). If one assumes that the ratio

$$\frac{l}{b} = 5,$$

which exists for high pile drivers (see Fig. 121), then the force in the supporting panel of the strap of the truss will be equal to

$$N = 5P.$$

connected with the enclosing of the cantilever at its base, were mentioned in this case to such a degree that it was necessary to introduce a conventional boom for sustaining the forces of lift into the design of a boomless cylindrical pile driver.

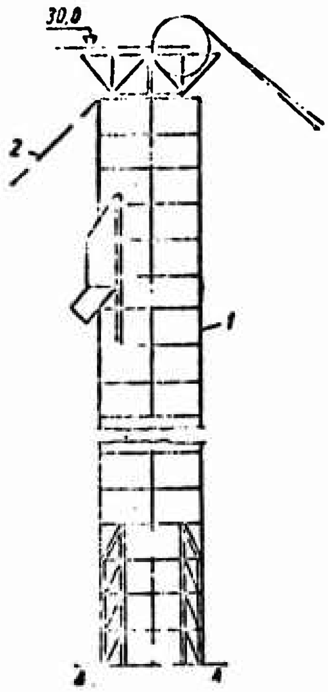

Fig. 134. Boomless cylindrical metallic pile driver: 1 - cylindrical mount (pile driver); 2 - possible position of the guy wire.

There is no need to dwell here on the description of cylindrical pile drivers with booms. One ought only to note, that these structures in essence are the particular variety of mounted pile drivers with an enclosed mounting of the annular section.

For ventilation stems the utilization of boomless pile drivers with enclosed machine tools can be appropriate, but the action of forces and the construction difficulties described above are an obstacle to the broad introduction of pile drivers of this design. In connection with this there is a proposal about the exception made from normative requirements according to the calculation of the deckhouse pile drivers in the case of the rupturing of the lift rope. This problem is very complex and is associated with the utilization

With a pulley 5 m in diameter and a rope 60.5 mm in diameter the breaking force will amount to 246 t, which corresponds to a value of P of about 470 t and to the amount N of more than 2300 t. If we compare the sums of the products of the forces in the rods along their length, then the ratio of these amounts for boomless and mounted pile drivers will be close to $\frac{l}{b}$, i.e., it will be equal, in this case, to 5.

The theoretical weight of the construction is proportional to the sums of the products of the forces in the rods along the length of the latter. In this way, the weight indexes of the boomless pile drivers in general are not favorable.

By bracing the boomless pile driver at its base one usually faces a number of complications mostly very considerable forces in the straps of the cantilever and in the anchor devices of the pile driver. Furthermore, in this instance is necessary a massive and stable collar of the mine shaft or a rigid foundation of the pile driver especially distributed at the base.

One of the examples of a solution of boomless (cantilever) pile drivers is a boomless cylindrical pile driver (Fig. 134), used in certain ventilation stems in the coal industry during the post-war reconstruction period. The mounting of the pile driver under the given conditions should be hermetically sealed, and the planking of the pile driver should possess a known strength. A number of subsequently installed metallic ferrules of the boomless cylindrical pile driver are substituted for the mount and planking of a simple boomless (cantilever) pile driver. Because of the cylindrical form, the thickness of the metallic sheets of the ferrules constitutes 6 mm. These circumstances help one to obtain in a given particular case, relatively more favorable indexes for the weight of metallic designs for a described cylindrical cantilever pile driver. Nevertheless, this design is characteristic for all the above described deficiencies which typify boomless pile drivers. The difficulties,

of safety and parachute equipment and other problems of safety technique. The proposal in this part, apparently, has not been founded. Without dwelling on this specific question, it is possible to note, that the efforts of researchers and of construction workers should be concentrated on improving the designs of boomless pile drivers.

The designs of cylindrical and other boomless pile drivers can be substantially facilitated and improved because of the introduction of guy wires.

With firmly embedded mounts in general in and specifically with *metallic boomless pile drivers* along with cylindrical and other machines is possible and a device of prestressed guy wire, 2 on the side, opposite the direction of the loads on the lift is expedient (Fig. 134).

Let the projection of the forces of the prestressed guy wire 2 (Fig. 135a), to horizontal P_1 causes a negative displacement of the machine (to the left along the axis of the abscissa) AA' equal $-\Delta$, and the negative moment basically of the console $-M_B$. In this case the bent axis of the centilever rod 1 (machine mount) is determined by line $A'B$, where A' - the top of the machine mount, B - the point of its enclosure at the base.

Let us apply a load P_2 as a result of the work of a lift to the top of the machine. If P_2 is equal to P_1, then the top of the machine will be located between points A' and A, but the value of the bending moment at the base of the machine between the values $(-M_B)$ and zero, respectively. If $P_2 = 2P_1$, then the top of the machine will be located between A' and A'', but the value of the bending moment at a base between the values $(-M_B)$ and $(+M_B)$, respectively. It is obvious that only with a further substantial increase in the load P_2, the value of the bending moment at point B will achieve a value $(+M_B)$.

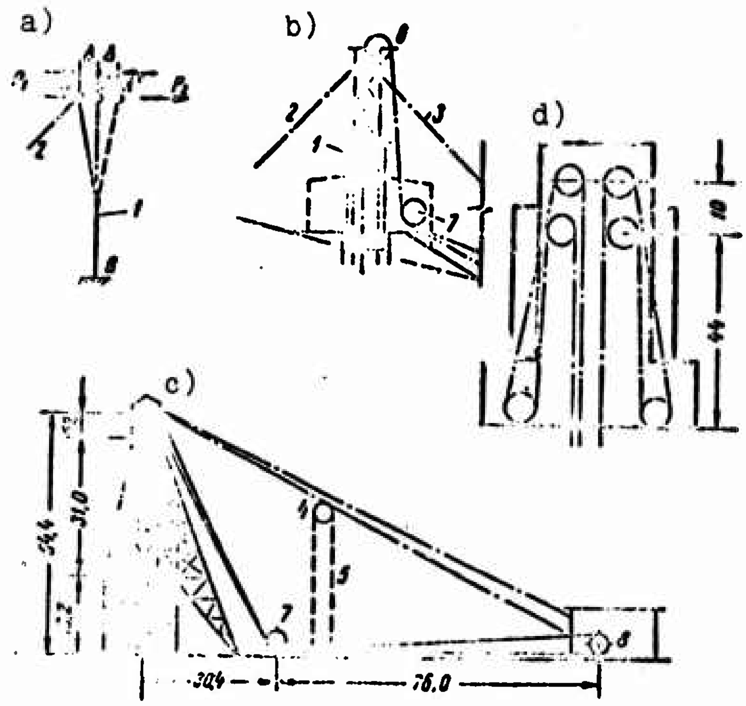

Fig. 135. The diagrams, which correspond to the minimum horizontal loads of the pile driver machine: Δ - displacement, corresponding to the force P_2.

In practice, if a metallic boomless pile driver with guy wires 2 (Figs. 135a and 134) is calculated, for example, for the action of load P_1, equal to 60 t, which accordingly to the data, corresponds to a conventional boomless pile driver having a weight of about 90 t, then it is possible to transfer a specific appearing load P_2, considerably greater than 120 t on this pile driver during the rupture of the lift rope, as a first approximation equal to 150-180 t.

If the value P_1 somewhat exceeds the effect of the horizontal component of the working load of the lift, then without considering the presence of the guy wire 2, it is possible to transfer the working loads of the lift on the given boomless pile driver; allowing for a prestressed guy wire 2, it is possible to transfer the specific loads of the lift to the pile driver, which appear as a result of the rupture of the lift rope.

On the basis of what has been proposed it may be concluded, that the utilization of boomless pile drivers possessing prestressed guy wires 2, directed to the side opposite the loads of the lift, using rigid enclosed or grated machine mounts deserves special attention.

The utilization of guy wires 2 and 3 together (Fig. 135b) with rigid machine mounts - is possible, and with highly flexible machine mounts - necessary. In the latter case the utilization of guy wires is appropriate with comparatively small loads and the permissible values of horizontal displacements at the tops of the machine mounting. The leads displacements at the tops of a machine mounting correspond to the least values of horizontal components of loads of the lift. Other conditions being equal, this corresponds to the approach of the lift machines to the stem of the mine shaft, i.e., mostly to a case of the ground position of the rope driven pulleys with a simultaneous device on the guy wire or a boomless pile driver of corresponding guide pulleys (Fig. 135b).

In certain cases obtaining the smallest values of horizontal loads of the lift, transmitted to the pile driver, and using single-rope lift devices with drum lift machines has not been excluded. Figure 135b depicts a diagram of the guy wire pile driver, which has special pulleys near its base, set in a collar and which serve for changing the direction of lift ropes taking into account the practical exception of horizontal external loads on the pile driver. A machine mount in this instance carries predominantly vertical loads of the lift, and the guy wires of the pile driver sustain comparatively small forces, specifically wind loads during the operation of the lift and at specific loads. In such a case using a limited section of the guy wires it is possible to guarantee the minimum horizontal displacements of the underpulley landing of the pile driver and a high degree of rigidity of the structure. One ought to show similar existing arrangements of pulleys from abroad. Figure 135c shows a rigid pile driver about 54 m high with the distance on a horizontal from intermediate pulleys 7, to the drum of a lift

machine 8, at about 76 m, and to the pulleys 6-30.4 m. The deflecting pulley 7 set at a level of planning near a pile driver replace the intermediate supporting pulleys 4 with a mast 5, for their maintenance.

The second way of using steel cables in pile drivers amounts to a *stretching device*, which secures the transverse stability of the structure. In this instance stretchings are not used or barely so in the direction of the loads of the lift and only the forces, which appear as a result of the effect of wind and seismic loads, are basically sustained.

The utilization of stretching in practice is limited so far to temporary structures during construction and heading. It follows, however, to show, that the utilization of steel cable stretching in the designs of a number of pile drivers is a simple and effective measure.

Inasmuch as in most cases it is possible to spreads the boom crosswise in metallic pile drivers without any difficulties, the utilization of stretching in metallic pile drivers can basically occur with high and very high pile drivers. As for reinforced concrete sectional ferroconcrete, wooden and other pile drivers, the utilization of stretching for these designs is even expedient with average height and low pile drivers. Sectional ferroconcrete pile drivers employing stretching can be used at practically any height. Transverse stretching permits a reduction of the number of the kind and dimensions of the elements of the sectional ferroconcrete designs of the pile driver and substantially reduces its weight. Transverse stretching in wooden pile drivers is also an effective measure. Under usual conditions the task of providing transverse stability of the most per fect diagonal and wooden pile driver mounts is most difficult to attain. In view of the difficulties, connected with the protection of the wood against atmospheric reactions, any kind of external boom, extending beyond the main flat design of the construction is undesirable. With the introduction of these booms in metallic designs difficulties

diminish but the overall expenditure of metal in the construction substantially increases. The introduction of stretching gives the same result, but with less expenditure of steel.

It is possible to show the following diagrams of the arrangement of transverse stretching for flat pile driver machine and others (Fig. 136):

diagrams *а*, *б* correspond to the haulage, parallel to the plane of the lift, stretchings perpendicular to the plane of the lift and the main plane of the pile driver;

diagrams *в*, *г* correspond to the haulage, perpendicular to the plane of the lift, the stretchings comprise by design with the plane of the lift at a certain angle, close to a straight line;

diagram *д*-- with haulage of unloading, parallel to the plane of the lift; the stretchings are perpendicular with the shown plane; the number of stretchings - 4; a diagram can be used with considerable dynamic loads of the machine, with large apertures in the latter, connected with the unloading of the skips of comparatively large capacity, and also in other similar cases;

diagram *ж* is possible for haulage, perpendicular and parallel to the planes of lift, stretchings located by design at an angle to the plane of the lift, with a component of 60° and more;

diagrams of *и*, *к* differ by the stretchings, arranged in the plane of the boom; in these cases a flat boom together with two stretchings perform the role of a conventional boom, set crosswise.

Some diagrams can be used for haulage, parallel and perpendicular to the plane of lift. Thus, for instance, diagram *ж* is an example of the possible organization of haulage in any direction. Diagram *a* can also be used for haulage, perpendicular to the plane of lift, with

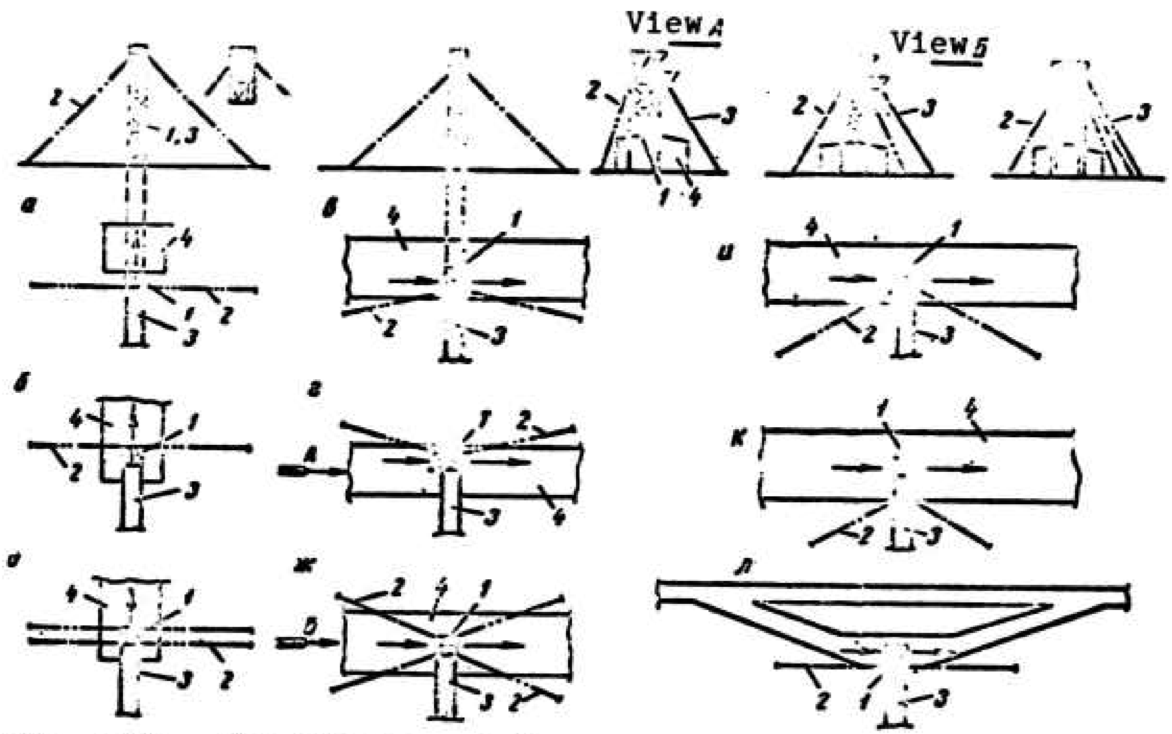

Fig. 136. The diagram of the arrangement of transverse steel stretching of flat pile drivers: 1 - machine mount; 2 - stretching; 3 - a boom; 4 - deckhouse building.

a definite outline of the galleries and haulage passages. A corresponding haulage is represented in diagram *д*. In this way, a very simple diagram *a* (it is, however, *д*) for the arrangement of stretching is rather universal from the point of view of the possible organization of the haulage.

One ought to note that the stretchings practically in all cases sustain the loads, acting in a direction, transverse to the plane of lift. These loads are primarily windy loads, and based on their absolute value, are small. Therefore, with comparatively light stretchings along with a very small total expenditure of steel cable in a flat pile driver it is always possible with stretchings to guarantee a small permissible value of horizontal displacements of the top of the pile driver in a transverse direction. On the other hand, the basic loads of the lift and other loads acting in the plane of lift sustain very rigid structures of diagonal or machine mounted flat pile drivers in this plane. Corresponding displacements affecting the loads of the lift in the given system are insignificant, which is the most important feature of flat pile drivers with stretchings.

The fastening of the stretching cables to the matching parts of the foundations at height of about 2 m above the level of planning is recommended, which is a guarantee against random damage to the cables (Fig. 137). The manufacture of stretching cables should be according to special instructions. In this case it is necessary to give special attention to the drawing up of cables with forces, somewhat exceeding the calculated forces and with markings taking into account the forces in the working position and the quality of the sealed off ends of the cables. The strength in closed ends should correspond to the strength of the cable.

Figure 133 depicts the diagram of a flat metallic ferroconcrete or wooden pile driver, the stability of which is provided crosswise for two stretching cables 1. The pile driver - flat-mounted at a height of 30 m. Steel cables 31.5 mm in diameter provide the

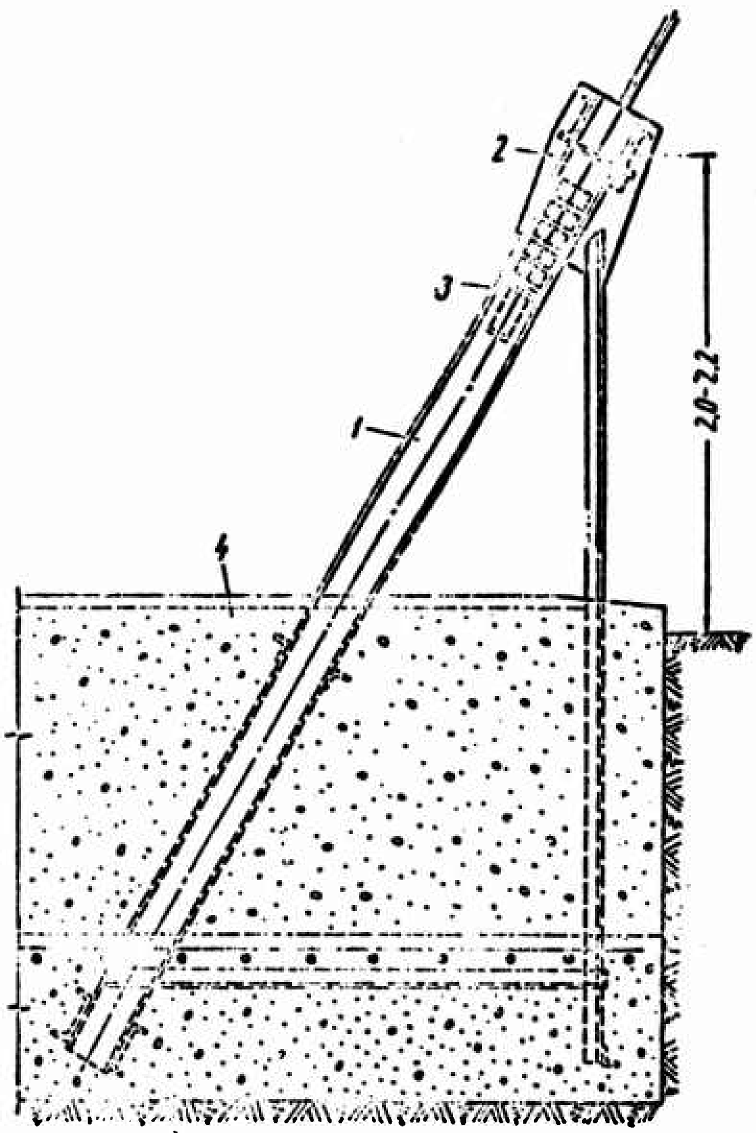

Fig. 137. The fastening of the stretching cable at a height of 2 m: 1 - metallic design; 2 - stop; 3 - parts of the fastening of the stretching cable; 4 - foundation.

stretching for the pile driver. The area of the section of all the wires of the cable is 5.89 cm^2, the length of the stretching is 4244 cm, the weight of the cable is 4.96 kg/m; the diameter of the frozen cable is taken equal to 6.3 cm; the weight of the ice is 2.1 kg/m. The sum total weight of the cables for the stays of the pile driver in the examined example is equal to 0.4 t, the weight of the metallic designs - 1 t (Fig. 137), the metallic components for the fastenings of the stretching cables are 0.4 t, the overall expenditure of metal constitutes about 1.8 t.

In accordance with the normative windy loads the horizontal reaction W_1 of the pile driver machine at point *B* (Fig. 138) constitutes 2.4 t. With the effect of the greatest possible calculated load of the wind, the corresponding horizontal response of the pile driver machine at point *B* - W_2 is equal to 4.4 t. During the calculation for the wind of the pile drivers with stretching cables one ought to consider a number of combinations of loads, including the effect of the normative load of wind W_1 allowing for the action of the wind on the stretching cable, the weight of the stretching cable and of the rime; the effect of the greatest possible calculated load of the wind W_2 allowing for the weight of the stays, the difference in temperatures. With the effect of the normative windy load, the amount of displacement at the top of machine is equal to

$$\Delta_1 \doteq 3{,}0 \text{ см} = \frac{1}{1000} H.$$

With the effect of the greatest possible calculated windy load the amount of the greatest displacement at the top of the machine is

$$\Delta_2 = 4{,}8 \text{ см} = \frac{1}{620} H.$$

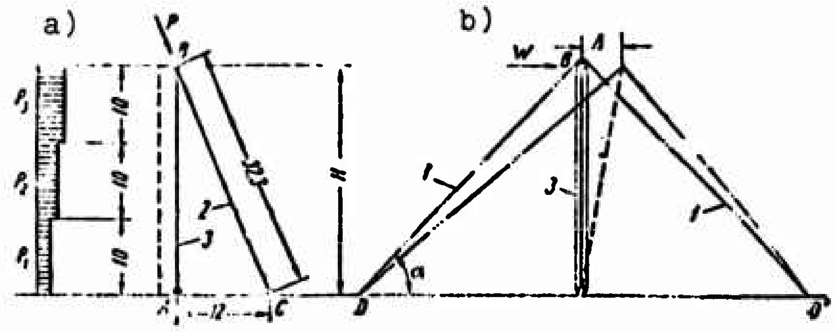

Fig. 138. The diagram of a flat-mounted metallic ferroconcrete or wooden pile driver, the stability of which is provided crosswise for the stretching cables: a) diagram of the frame of the pile driver in the plane of lift; b) diagram of the structure in the plane, perpendicular to the plane of lift; 1 - stretching cables; 2 - boom; 3 - mount.

The obtained amounts of displacements $\left(\frac{1}{1000}-\frac{1}{620}\right) H$ are entirely acceptable for cargo lifts. For cargo-passenger lifts, the magnitudes of displacements at the rated load of not more than $\frac{1}{800} H$ is recommended.

It is interesting to note that for the comparison of the values of the displacements at the top of a rigid metallic pile driver with a boom are distributed crosswise. A rigid pile driver machine close based on sizes was examined in Chapter V. The height of this pile driver was 29.9 m; the length of the boom, 34 m, and the area of the section of the strap of the boom, 153.6 cm^2. The displacement at the top of a rigid metallic pile driver machine with a boom at a normative load constitutes 1.2 cm, or $\frac{1}{2500}H$, and at the largest possible calculated load - 2.2 cm, or $\frac{1}{1350}$ H.

4. Ferroconcrete Pile Drivers

The known designs of ferroconcrete pile drivers are characterized by considerable weight. Therefore, one usually avoids transferring their weight to the shoring of the stems of mine shafts. At the same time with the increase in the weight of the construction, the direction of the general resultant loads of the pile driver approaches a vertical line, the designs of pile drivers, to a lesser degree, operate under bending, and the pressure in the plane of the base of the construction based on their magnitude, are relatively isometric. These and other motives are proposed as the basis of a number of designs of boomless ferroconcrete pile drivers, in a number of cases, united with adjacent deckhouse construction.

The pile driver, joined with the designs of receiving hoppers and deckhouse building (pile driver hopper), used in the coal industry, are shown in Fig. 139. Fig. 140a shows a plane of a boomless (cantilever) ferroconcrete pile driver, made for conditions at Whitewatersrand. The outlines of the pile driver according to this plan differ somewhat from that mentioned above. Nevertheless, all the listed construction are boomless (cantilever) pile drivers, associated, in most cases, with the support designs of deckhouse buildings and characterized by considerable weight. One of these ferroconcrete monolithic pile drivers with a built-in ferroconcrete receiving skip hoppers (pile driver hopper) with a height of the construction at 40 m and expenditure of monolithic reinforced concrete of 1900 m^3 has an overall weight of about 7000 t. In other words, such a pile driver hopper with a height of 50.5 m is characterized by a pressure in the plane of the base of about 10,000 t. The weight indexes of a number of boomless ferroconcrete pile drivers with guide pulleys are close to the weight indexes of ferroconcrete tower pile drivers, which support cable-drive pulleys of multirope lift devices.

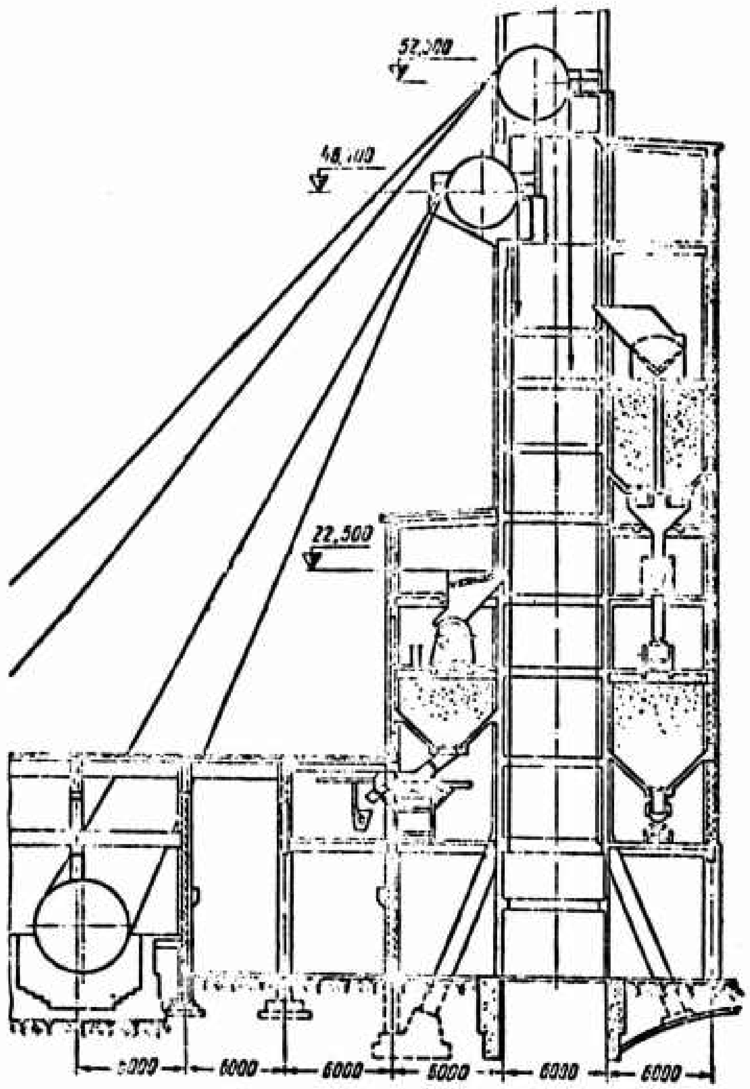

Fig. 139. Boomless ferroconcrete monolithic pile driver, joined with receiving hoppers and a deckhouse building (pile driver hopper).

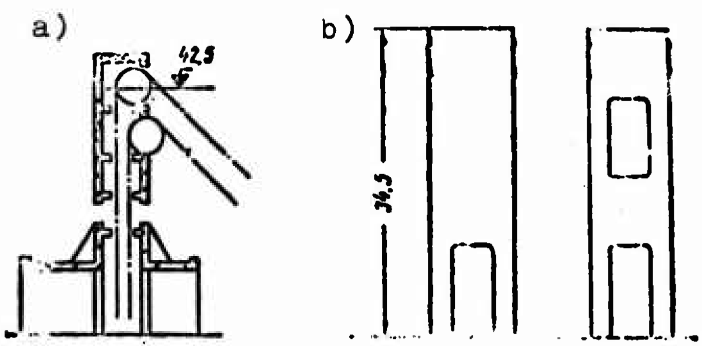

Fig. 140. Ferroconcrete boomless pile drivers based on the data of a foreign practice.

The need for the erection of construction, which is characterized by a considerable volume of monolithic reinforced concrete near the stem of the shaft, leads to difficulties in the building of the central mine unit and to corresponding investments of time which substantially eliminates the period of the operation of the pit. One ought to keep in mind the latter during the examination of the problem about the utilization of monolithic ferroconcrete pile drivers. In all such cases it is necessary to take measures which facilitate the acceleration of the construction work, specifically, problems should be examined about the utilization of mobile or adjustable planking, about the form of construction, to the greatest degree, which corresponds to the outlined production of the construction work. Figure 140b shows a ferroconcrete boomless pile driver with guide pulleys, designed for the utilization of mobile planking. The ferroconcrete box of one of the pile drivers of this type with a height to the underpulley landing of 31.5 m was also erected in 10 days was using mobile planking.

Among the other monolithic ferroconcrete pile drivers one ought to dwell on the description of A-frame, triangular-hipped and polygonal-hipped systems. These pile drivers do not bear down on the collars of mine shafts, and their foundations are spread by design. The machine mountings for the bracings of the guide and barrier of the lift are usually made of metal. The frame of the construction – monolithic, ferroconcrete, with heavy sections of rods. Utilization of A-frame and other listed designs is extremely limited (the reasons were indicated above). The basic deficiency of such ferroconcrete pile drivers also consist of a number of complications, connected with the fulfillment of monolithic rod designs of high construction, located in the congested central mine unit. Furthermore, the time necessary to erect such a ferroconcrete pile driver, is quite long in comparison with other variants of the solution of the designs of a pile driver.

The more or less wide utilization of ferroconcrete pile drivers can be obtained with the modernization of the above described designs, directed towards their simplification, reduction in the volume of the

ferroconcrete work, produced at a considerable height, reduction of the expenditure of wood for scaffolding, planking and reduction in the weight of the construction.

One of the solutions, specifically for boomless pile drivers, would be the utilization of reinforced concrete structures with a metallic head and a ferroconcrete supporting part with the introduction of supporting reinforced frames, the utilization of which in a number of cases is expedient in pile drivers for the following reason. Inasmuch as temporary loads in pile drivers, including the specific loads, which correspond to the rupture of the lift rope, are quite great, and the constant loads, determined basically by the weight of the skeleton construction, are comparatively insignificant, the number of rods of the construction can be small. The sections of rods have large sizes with simple rectangular forms, which is favorable for the utilization of supporting reinforcement frames.

The part of construction most highly located and represented by a combination of comparatively complex elements of the head of a pile driver, can be made exclusively of metal. This will somewhat increase the expenditure of metal, but will substantially simplify the erection of the pile driver and will permit the weight of the construction to diminish sharply.

A diagram of the skeleton of the reinforced concrete boomless pile driver with receiving hoppers (pile driver hopper) is given in Fig. 141. The upper part of this construction in composition of the head and supporting adjacent design *1* is represented by a metallic pile driver of a boomless (undeveloped triangular-hipped) system. Actually this part of the pile driver is the conventional head of a large pile driver machine (see Fig. 114), simplified in its lower part because of exceptions which correspond to the underpulley and transverse trusses. The monolithic ferroconcrete design *2* is applied in the lower part of construction approximately within the limits of the usual height of a deckhouse building, which is most

convenient in relationship to the organization of the lift and laying of concrete as well as in conjunction with the utilization of the supporting reinforcement frames of the reinforced concrete part of the pile driver and of the above described metallic upper part, and the possibility of erecting the pile driver in short order is assured.

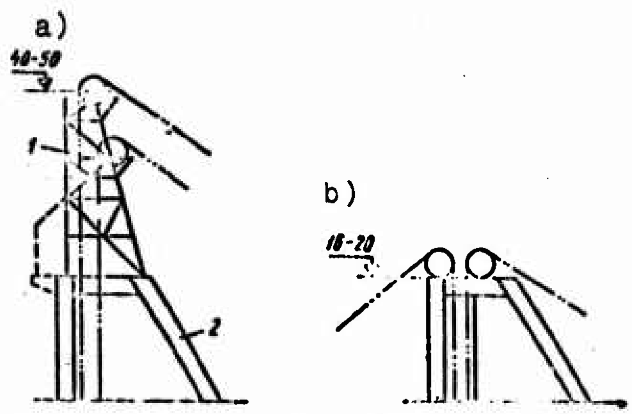

Fig. 141. Reinforced concrete skeleton boomless pile driver with a receiving hopper (pile driver hopper): a) the diagram of the pile driver; b) the diagram of the utilization of the lower reinforcement framework part of the pile driver during driving; *1* – metallic part of the pile driver with the head and supporting structures; *2* – the ferroconcrete part of the frame of the pile driver, in operation with the supporting reinforcement frames.

The manufacture of the head of a pile driver using reinforced concrete is most intricate and laborious. Successful contemporary solutions for this actually do not exist. Therefore, the described diagrams of reinforced concrete pile drivers (Fig. 141) can be, in a number of cases, accepted for operation. The expediency of this diagram will be most clearly expressed with an average height of the pile drivers and with a moderate height of the lower section of the construction, made from reinforced concrete using supporting reinforcement frames. The supports of such a pile are arranged outside over-all size of shoring of the stem of the mine shaft, allowing for the over-all size of their sinking construction and their displacement over a certain period of time inside the permanent pile driver. Such a pile driver can also be used for driving the stem, whereby the lower ferroconcrete part of the pile driver is also

installed and simultaneously cemented with the equipment of the ferroconcrete shoring of the mouth of the shaft stem, whereupon the sinking landing is equipped with the installed required pulleys (Fig. 141b). Based on the termination of the driving or simultaneously with it, it is possible to install the upper metallic part *1* of the permanent pile driver.

Sectional ferroconcrete pile drivers. Pile driver machines are one of the most commonly used systems of metallic pile drivers. During the building of ferroconcrete pile driver machines there are a number of serious difficulties, especially noticeable using precast reinforced concrete. The ferroconcrete machine mount on the shoring of the mouth of the stem shaft is far from always being acceptable; the booms of pile driver machines differ by great length, which, with considerable loads, is equivalent to the utilization of ferroconcrete elements of a boom with a weight corresponding approximately to the weight of bridge span structures. The number of rods is so small that the repeating of the elements is insignificant. With metallic designs because of the introduction of bracing, which transmits comparatively small forces from the weight of the boom itself to the machine, the sections of the boom can be substantially reduced. The weight of the ferroconcrete boom is calculated in tens of tones, and to transmit such forces of the structure of the sectional frame machine is difficult.

The support of the metallic pile driver machine simultaneously in the collar of the mine shaft and on a separately standing foundation of a boom is possible namely because of the utilization of metallic structures. The utilization of the ferroconcrete pile driver machine, which is characterized by a rigid joint of the boom and macnine, requires the introduction, in a number of cases, of special foundations under the mounting, substantially spread by design. Thus, the need is created to use a machine apart from the vertical and horizontal elements of the sloping rods, which transmit the load to the foundations of the machine. Everything shown, on the whole, substantially complicates the realization of the idea of utilizing the sectional ferroconcrete pile drivers of a machine system.

Schemes are known of sectional ferroconcrete pile driver machines developed with the utilization of the sectional principle. The schemes encompass small pile drivers with a height of 15.1; 19.9; 20.5 and 24.7 m, supporting pulleys with a diameter of 2.5 m (Fig. 142a). A pile driver with a height of 24.7 m is composed of eleven sections of the machine, the structures of the head and five sections of the boom. The stressed reinforcement framework of the designs of the boom is provided. The deficiency of the described diagrams are the great lengths of booms, significant moments in the rods of the booms and the need for their stressed reinforcement, although these rods with the effect of working and special loads of the lift sustain compressive forces. Also one should relate the presence of the sections of a machine to the deficiency of the diagrams. With a certain change in the section of a machine the overall size of the ferroconcrete sections of the machine also changes.

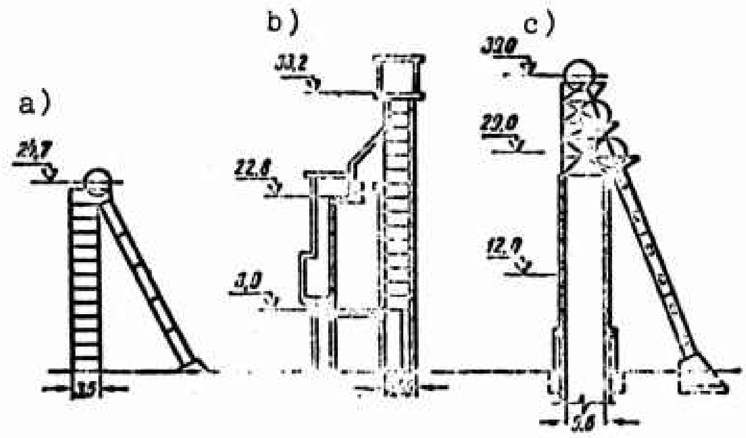

Fig. 142. Sectional ferroconcrete pile drivers: a) sectional ferroconcrete pile driver machine; b) sectional ferroconcrete boomless pile driver joined with the receiving hopper; c) sectional reinforced concrete pile driver.

Figure 142b shows a diagram of the sectional ferroconcrete pile driver, used in the copper-ore industry. At a full height of 38.2 m this pile driver, up to a level of 22.8 m, is associated with the

ferroconcrete monolithic receiving hopper adjacent to it. The wall of the machine, the parallel planes of the lift, are formed by the elements of a double-T form at a height of 2.1 m, width and the thickness of the wall of a double-T, 3.4 × 0.2 m, a section of the flange, 0.97 × 0.3 m.

Figure 142c depicts diagram of one of the deckhouse pile drivers, provided by a standard deckhouse complex of an iron-ore mine. This pile driver differs in the design of the head, which is metallic. The design of a boom here is somewhat simplified because of the unification of the rods of the boom into rigid flat frames. The deficiency of a solution continues to be the large overall length of the boom of the pile driver, consisting in this case, 27 m.

The positive results with the introduction of precast reinforced concrete in the design of deckhouse pile drivers can be obtained, specifically, on the basis of the realization of the diagrams of boom-upright reinforced concrete sectional pile drivers.

The external form of such a pile driver suggests a trestle of an inclined lift (Fig. 143). The head of the pile driver *5* – metallic. The pulleys are installed on metallic structures, which consist of a continuation of the ferroconcrete boom *2*. The equipment of the head is similar to that given in Figs. 116 and 117. The ferroconcrete uprights *1* are the support of the ferroconcrete boom *2*, and together with it, the joints *3* and cross pieces *4*, form an invariable very rigid system in the plane of the frame of a pile driver. The boom consists of separate repeating elements having a length of about 13-14 m. The great length of the elements corresponds to the bracing of a boom crosswise, usual for rigid pile drivers. The amount of bracing of the boom in this case is somewhat less in comparison with the requirement for metallic pile drivers. The lesser length of the elements corresponds to a boom with parallel straps. Similar flat ferroconcrete booms can be used with wide machines including the average pile drivers based on height.

The unification of boom-upright pile drivers is, by comparison, easily accessible. With a small number of the type and dimensions of sectional elements it is possible to obtain a construction of different height and purpose. Specifically, unification is possible with the arrangement of pulleys in one vertical plane and haulage crosswise. The three sectional elements described above (at height modulus of 12 m) and four type and dimensions of metallic heads are quite sufficient for the formation of a series of boom-upright pile drivers having a height of 15; 18; 21; 24; 27; 30; 33; 36 m and so on. With high pile drivers the utilization of the modulus of the height of construction, equal to 18 m and more is possible.

5. Wooden and Other Pile Drivers

The utilization of wooden pile drivers is limited because they are a fire hazard, have comparatively low strength and structural complexity in the presence of several lifts. Nevertheless, in many enterprises, they use and design wooden pile drivers, especially in forested regions. It is also possible to note, that their utilization is relatively developed in the iron-ore, copper-ore, manganic and gold-prospecting industry.

The utilization of wooden pile drivers in a number of cases is expedient in new remote regions, in separate small undertakings, all which results in a substantial economy of the metal and it facilitates the acceleration of building and delivery of a mine or part of it into operation.

The above described systems of pile drivers are most clearly defined in the fulfillment of the structure made of metal. However, the above mentioned classification of systems of pile drivers are applicable even for wooden pile drivers. The latter are also divided into hipped (in this case predominantly polygonal-hip), semi-hip, machine, diagonal, guy wire systems.

There are designs of pile drivers, in which the separate parts and parts of pile driver are made using metallic elements. The latter can be presented by metallic drawbars, stretchers and guy wires, separate landings and joints, half-booms and booms. The utilization of metal in such designs is limited, and pile drivers and their units retain the basic distinctions of wooden designs. Therefore, wooden-metallic and some other pile drivers will be examined together with wooden pile drivers.

According to the rules of safety wooden pile drivers should be equipped with a sprinkling system.

Hipped wooden pile drivers. Hipped wooden pile drivers frequently have the form of frustum (Fig. 144a), developed in a horizontal direction to the side of the lift machine with face *CD*, directed approximately along the resultant forces of the lift. As a result of the tendency for the utilization of elements of crosspieces and bracings as short as possible frequently intermediate uprights *CM* are inserted, and with a considerable size of the construction – two such uprights, *CM* and *CN*. The panels of the vertical planes of the pile driver are filled with the grating of various outlines, and in horizontal planes, *A'*, *A"*, and others the joints which facilitate the stabilities of the elements of the external planes of the pile driver are inserted.

The underpulley landing *BC* is usually represented by a crib of braces uprights, composed of underpulley and transverse beams. In rarer cases the underpulley landing is formed by grated wooden-metallic designs, and sometimes metallic designs in the form of a beam cage or trusses.

The external planes of the pile driver during comparatively long use of the construction are faced with corrugated asbestos-cement sheets on wooden lathing, and in other cases – by boards.

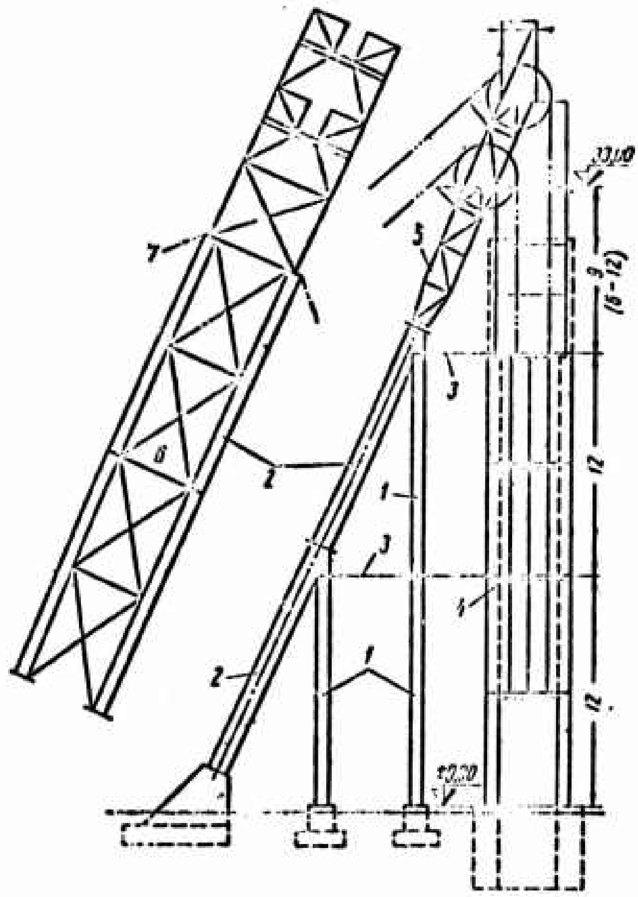

Fig. 143. The diagram of a boom-upright of a sectional ferroconcrete pile driver.

The stability of boom with parallel straps and the pile driver, on the whole, is secured crosswise by metallic joints *6* and by analogous transverse joints in the planes of the uprights *1*. The transverse stability of high pile drivers can also be provided by transverse stretchers *7* from steel cables. Joints can be ferroconcrete frames.

The mount of the pile driver sustains comparatively small forces. Therefore, the frame of the machine is represented by lightweight elements of small sections, frequently by uprights *1*, which can be, in most cases, set on the shoring of the stem of the mine shaft and

connected in a transverse direction. The stability and the rigidity of the machine in the plane of the main frame is assured by using joints *3* and cross pieces *4*, and also by rigid unions of the sectional elements of the machine in the joints of the frame.

It is expedient to use a united vertical modulus for the designs of the pile driver. Specifically, the height of all ferroconcrete uprights and the height of the sections of a boom can be taken equal to 12 m. This secures the rigidity of the system in the plane of the main frame, the minimum number of unions of uprights and the possibility of introducing of the standard ferroconcrete wall slabs, ribbed or plane, with nominal sizes, 6.0 × 1.2 (0.6) and 12.0 × 1.2 m, into the external planes of the machine. These slabs, joined by welding through matching parts with each other and with the elements of the machine and unitized, form rigid discs in the main and transverse planes. Not excluded is the equipment with the confines of these discs of metallic frames made of horizontal cross pieces and angular vertical elements, with an exception in this instance, of ferroconcrete uprights and cross pieces of the machine.

In this case or another the total given expenditure of reinforced concrete per 1 m^2 of surface of the walls of the machine amounts to 0.05-0.1 m^3, which corresponds to the weight of the average machine of 50-100 t. According to the absolute value, this exceeds the weight of a metallic machine, but one ought to take into account, that in this case, the mounting hardly sustains the load of the lift. In this way, a load on the shoring of a stem with a boom-upright ferroconcrete pile driver is approximately equal to the load corresponding to a metallic pile driver machine. Therefore, in the overwhelming majority of cases, the boom-upright pile driver machines can be directly installed to the shoring of the mouths of the stems of shafts. One ought to note, however that with a hinged union of the joint *3* with the joints of the boom and machine a somewhat different settling of the boom and machine (shoring of the mouth of the stem) hardly reflects the strength and stability of the construction.

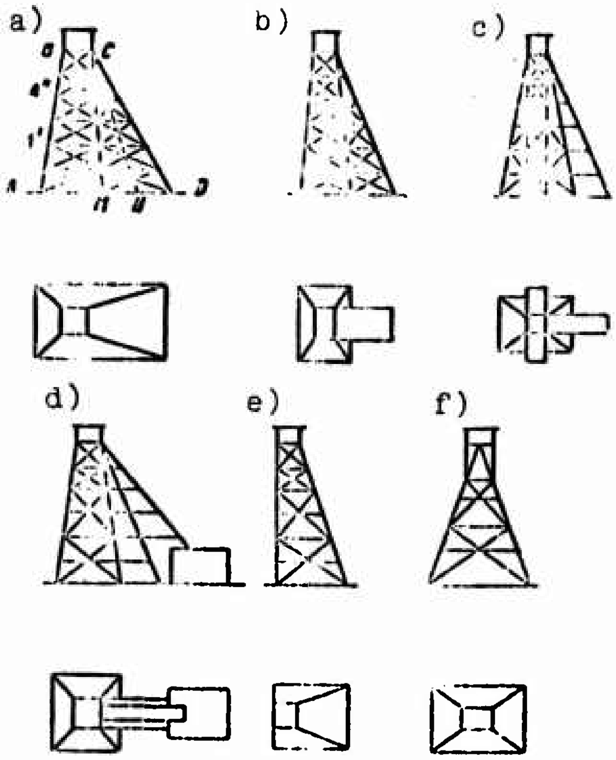

Fig. 144. The diagrams of wooden hipped pile drivers: a) form of the frustum; b, c) with booms, having parallel chords of the trusses; d) in the absence of a rupture between the pile driver and the machine housing; e) with a vertical front face; f) with two machine housings, arranged on the opposite sides of the pile driver.

The variations of forms of hipped pile drivers are presented in Fig. 144b, c, d. The relatively frequently applied form in Fig. 144b is obtained during the development of a section of the boom only over a part of the width of the construction, which usually corresponds to a width of the underpulley landing. The boom, using this form, has parallel straps, which simplifies its manufacture.

The rarely applied form, shown in Fig. 144d, corresponds to a case, when there is no rupture between the pile driver and the machine housing. Form e has a vertical front *AB*, which results in the reduction of the length of the elements of the grating and the simplification of the joints of the front truss of the construction.

Form f, where the triangle, formed by the booms with the base is used for the basis, it is suitable during certain lifts with an arrangement of the lift machine or winch from the opposite sides of the pile driver.

For the benefit of selecting forms a and b the following motives are given: the possibility of placing all the equipment in one construction and the absence of the need for a special deckhouse building. With haulage at the level planning the given reasons diminish, since the cost per unit of volume of the pile driver is significantly higher in comparison with the cost of a one-story deckhouse building. In general, when selecting a pile driver one ought to make a comparison of the cost and the deckhouse building according to variants.

Haulage in a hipped pile driver is usually directed parallel to the plane of lift, which can be explained by the simplicity of the equipment, in this instance, the underpulley landing of the wooden pile driver. The unloading of the skips is usually produced also parallel to the planes of the lift, sometimes through breaks in the front truss of the pile driver.

The receiving landings, tied in with the adjacent haulage galleries are arranged at the necessary level for the method of mine trolleys.

For the bracing of the rail guides in hipped pile drivers special wooden mounts made from uprights, joints and buntons are usually installed, which correspond to the arrangement of the buntons in the stem of the mine shaft. The underpulley landings are equipped with devices for the lift and installation of the pulleys, which usually consist of metallic coiled beams, suspended in the plane of the pulleys to the wooden designs of the hip of the underpulley landing.

Figure 145 shows a hipped pile driver having a height of 20.5 m. The pile driver has a simple grating. In this case the elements of the booms, as a rule, are not crossed and are located predominantly in one plane. Sometimes, these intersections are obtained because of the different number of elements of the bracings. The number of uprights of a main truss is small, the outlines of the rear semicantilever truss, which is used with parallel straps, are very simple. The underpulley landing is a beam cage made of wooden bracings with cuts as quarters of the wood. The mounting does not have bracings and is also composed of vertical uprights and horizontal buntons, connected by simple notches and clamps or drawbars.

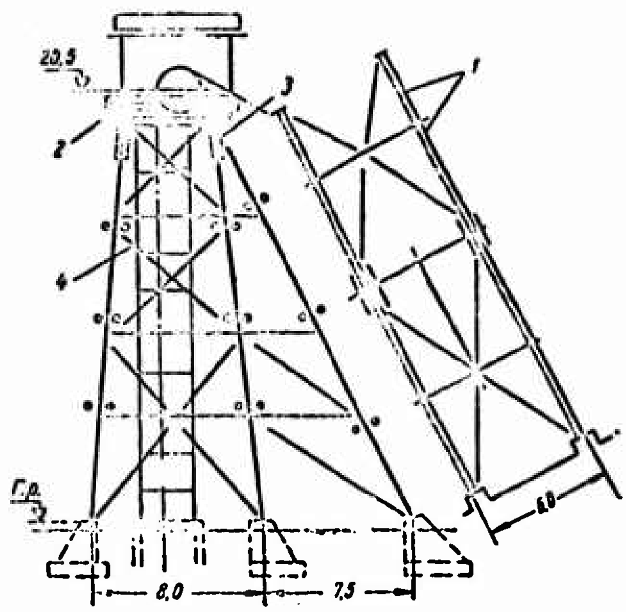

Fig. 145. Wooden hipped pile driver having a height of 20.5 m: *1* – the strap of the boom from the log having a diameter of 280 mm; *2* – underpulley and transverse beams from bracings with rough edges, 240 × 240 mm; *3* – main joint; *4* – mount.

Designation: Г.р. = horizontal buntons

The important distinction of the described pile driver is the simplicity of the main joints, in which there are no splines and complex parts. The presence of the frontal notches secures the unfailing work of the joints with the usual operation of the joints of wooden designs (see Fig. 151).

With the solution of the underpulley landing in the form of a simple beam cage made of wooden braces sometimes there are a number of difficulties, especially with large loads. The observed bulkiness of the underpulley landing and the investment of wood, in this instance are considerably reduced during the partial utilization of substitutes for wood, specifically metal. Figure 146 shows a diagram of a hipped pile driver having a height of 26 m. The pile driver supports two pulleys with a diameter of 2.5 m. Metal is used for underpulley landing equipment (Fig. 147). By using metal the supporting joints of this construction can also operate. Without going into details, it is possible to show that the local utilization of metal in wooden pile drivers having large diameters of pulleys, and in other difficult cases, allows one to solve the problem of the equipment of heavily loaded joints.

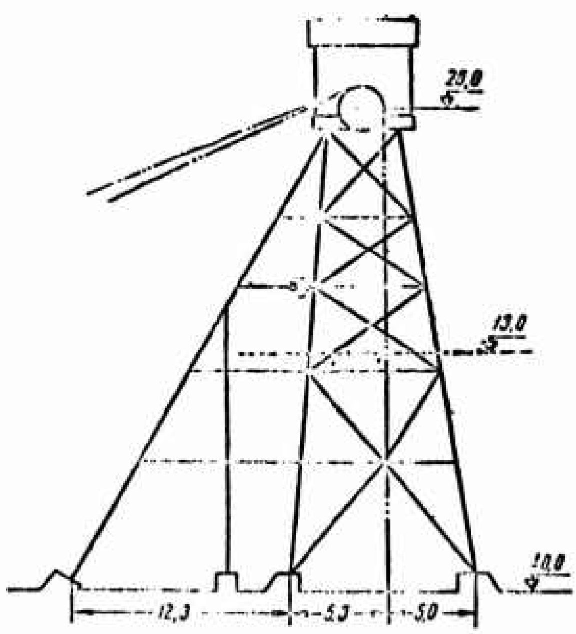

Fig. 146. Diagram of a hipped pile driver.

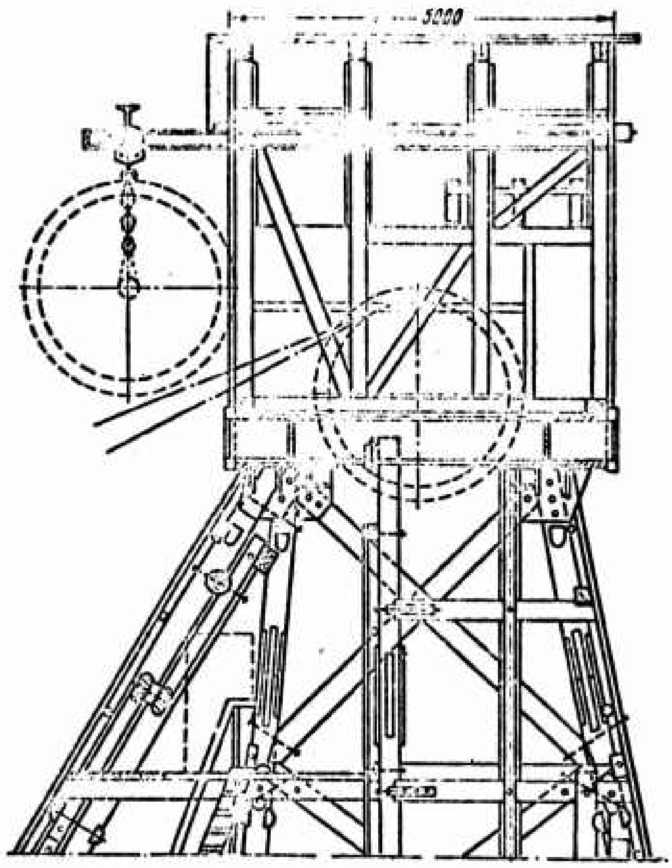

Fig. 147. The union of the metallic underground landing with the wooden frames of the pile driver.

The amount of expenditure of wood in hipped pile drivers can be found from the formula, analogous to the formula of the weight of metallic pile drivers. The volume of wood

$$V = aH\sqrt{P_{oc}},$$

where V – the volume of wood of the pile driver, m^3; H – the height of the pile driver, m; P_{oc} – the value of the rupture force of the cable, t.

The character of change in the coefficient a can be clarified by the graph, presented in Fig. 70. The values of the ordinates in this case will be otherwise, but the value of the coefficient is also

diminished with an increase in the values of the rupture forces of the cable.

With small values of rupture forces (up to 15 t), which corresponds to pulleys having a diameter up to 1.6 m, the coefficient *a* fluctuates within the limits of 0.9-1.4, and on the average, constitutes 1.2. For pulleys, 1.6-2.0 m (with rupture forces of the cables, 15-25 t) the coefficient *a* on the average, is equal to 1.1; for pulleys, 2.5-3.0 m (with rupturing forces of the cables 40-60 t) – about 0.9. With rupture forces more than 80 t, a further lowering of the value of the coefficient is observed.

Half-hipped wooden pile drivers. Hipped pile drivers are characterized by a relatively high expenditure of wood in its body. In connection with this one ought to pay attention to the comparatively little known schemes of wooden pile drivers of the half-hipped systems, which are substantially lightened and simplified hipped pile drivers.

A half-hipped wooden pile driver is characterized by diagram shown in Fig. 148a. Here *ABCD* – the body of a pile driver, where the central parallelepiped *MNPT* corresponds approximately to the overall size of the machine, *PRST* – the boom, in most cases, plane and with great large length, reinforced by auxiliary uprights in the form of a transverse frame, *FG*.

The described above solutions of simple joints of hipped pile drivers are quite applicable even to half-hipped pile drivers. The grating of the latter, especially in planes *AB*, *CD*, *NP*, *MT*, should be simple and clear without an intersection of bracings in one plane. Transverse frames *AB* and *CD* should be rather rigid in the lower part, which can be guaranteed by the utilization of a rigid cross piece *1* (Fig. 148a), supporting bracings, and also span beams, *AB* and *CD*. The joints of the pile driver can be accepted as in Fig. 151.

The sheathing of the pile driver is done according to the outline, *ABCD*. The boom is covered with lightweight roofing on the lathing in a plane, *PRTS*, with the boards predominantly according to diagram b, and sometimes according to diagram c. In a number of cases it is necessary to extend continuous sheathing also on a plane, *PRTS*, i.e., fully enclose the pile driver. Local shelters, in all cases, ought to be referred to, attentively, and it is necessary to provide a qualitative made shelter with the lap of the roofing not less than 200-300 mm lower than the overall size of the rod under the condition of the simultaneous realization of the ventilation of all surfaces of the wood.

Half-hipped pile drivers with a reduced boom, half-boom (Fig. 148c, d) differ in the arrangement of the boom, directed from point *S* not to point *T*, and at joint *M*, which is solved rather simply (see Fig. 151c), and the most important joint of a pile driver is in these diagrams. At joint *T'* the direct transmission of the forces from rod *MT'* to rod *TS* is usually secured. The latter, specifically, is possible with symmetrical cover plates on the bolts, subtending the shown two rods and the upright, *TD*, which in the given joint, is not interrupted, With double elements *T'S* the junction plate of rods, *MT'* and *T'S* can be arranged even more simply – by means of a direct overlap of element *MT'* and upright, *TD*.

The length of a boom in its reduced arrangement in a half-hip pile driver can be substantially reduced. The utilization of a half-hipped pile driver with a reduced boom is quite expedient with the relatively wide body of the pile driver *MT*, with a small and average height. Furthermore, if it is necessary to use a relatively steep boom, consisting of small angles with the plumb-line, transitional to a reduced boom (to a half-boom) considerably better angles of abutment are assured, which has special importance for wooden designs.

At a reduced arrangement of the boom and its small length it is frequently expedient to use metallic half-booms; thus, for instance,

in diagram c (Fig. 148) the boom in section $T'S$ can be made from two separate metallic rods without joints. A similar very simple metallic half-boom of small length is frequently simpler for manufacture in comparison with the normal full-size wooden boom and simplified boom TS of a half-hipped pile driver according to diagram a (Fig. 148).

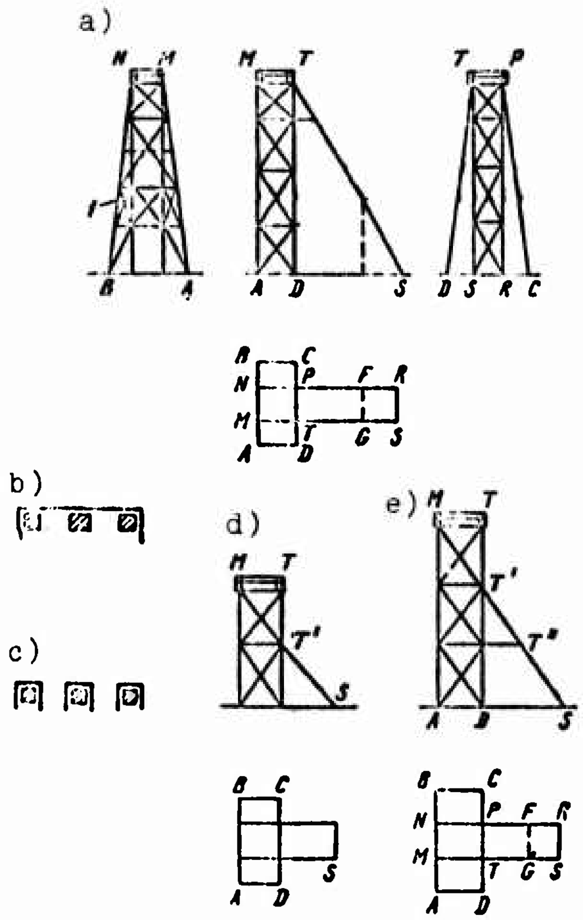

Fig. 148. The diagrams of half-hipped wooden pile drivers: a) frame of a half-hipped pile driver; b, c) local shelters of the wooden elements of the boom of a pile driver; d, e) half-hipped pile drivers with a reduced boom c) and with half-boom d); *1* – rigid cross piece of a transverse frame.

Pile drivers with a reduced boom are faced along the outline *ABCD*, the boom $T'S$ is covered, as shown in diagrams b and c (Fig. 148). The metallic half-booms are stained with oil paint using two coats.

One ought to pay attention to the features of the utilization of the half-boom, the presence of which creates the need to introduce elements, supporting the joints of the boom on the rear (transverse) truss of the half-hipped pile driver in the plane *CD*. These elements can be vertical or horizontal. The horizontal elements – are simpler and can be combined from one of the cross pieces of the frame of the pile driver. One also ought to have in mind that cross piece *MT* (Fig. 148d) sustains considerable tensile forces, which should be sustained in joint *M*. At the joint *T* to the rear truss of the pile driver considerable vertical loads are transmitted, the consequence of which this joint, as well as joint *M*, should have reliable simple notch-stops and tension devices.

Half-hipped pile drivers with booms and half-booms in comparison with corresponding hipped pile drivers require considerably less expenditure of basic material. The expenditure of wood in the body for half-hipped pile drivers approximately constitutes

$$V = (0{,}6-0{,}7)\,H\sqrt{P_{oc}},\ \text{м}^3.$$

Half-hipped and hipped pile drivers do not rest on the shoring of the mouth of the stem of the mine shaft. The foundations of these constructions most frequently are separately standing concrete or ferroconcrete monolithic and sectional columns.

The expenditure of metal, necessary for the manufacture of bolts, belts, washers, cover plates, of segments of rolled profiles constitutes 30-60 kg/m^3 of the wood in the body. Lesser figures pertain to hipped pile drivers. The expenditure of metallic articles and parts on one hipped pile driver with a height of about 20 m, and frequently constitutes 3.5-4.5 t. The expenditure of metal during the manufacture of any joints from metal (for example, the joints of the underpulley landing) increases.

Machine mounted and other wooden pile drivers. The diagram of the wooden pile driver of machine-mounted system differs somewhat from the usual scheme of a metallic pile driver machine. Thus, for instance, the bracings of the wooden boom is not always suitable With an increase in the width of the boom the sizes of the protective coating against atmospheric moisture increases respectively, which leads to an increase in wind resistance and it approaches the design of a hipped pile driver. On the other hand, the transverse bracing of the boom, made comparatively simply with a small height of pile drivers, is considerably hampered with an increase in height, i.e., when it is especially necessary. In connection with this the transverse stability of machine mounted and diagonal wooden pile drivers usually is assured because of the necessary broadening of the machine.

Figure 149 shows diagram of a wooden machine mounted pile driver. The height of the construction is 17 m, the guide pulleys have a diameter of 1.6 m. The haulage is done parallel to the planes of lift at heights of 7.5 m above the level of the surface. The mounting of the pile driver is composed by five standardized trusses with parallel straps, from which the three central ones are in the planes of the buntons of the mine stem. The rail guides are fastened directly to these three trusses. The horizontal spacers of the trusses are simultaneously the buntons of the machine mount. The lower sections of the uprights of the chords of the trusses at a height to 6 m are reinforced with additional uprights, supporting the underside stop beams of the receiving landing. The underpulley landing consists of underpulley and transverse beams made of wooden braces, connected with the help of very simple retangular notches. Stairs on the underpulley landing are placed in one of the compartments of the machine.

The boom consists of three fraces, arranged in the planes of the three central trusses of the machine mount. Cover plates necessary for the transmission of forces to the adjacent structures of the underpulley landing and to the foundations of the construction

are provided for the uprights of boom in supporting and upper joints. Besides the horizontal joints the boom has diagonal braces made of metallic drawbars. At two levels the boom is connected with the elements of a machine mount with cross pieces, in the plane of which horizontal lightweight metallic joints are located.

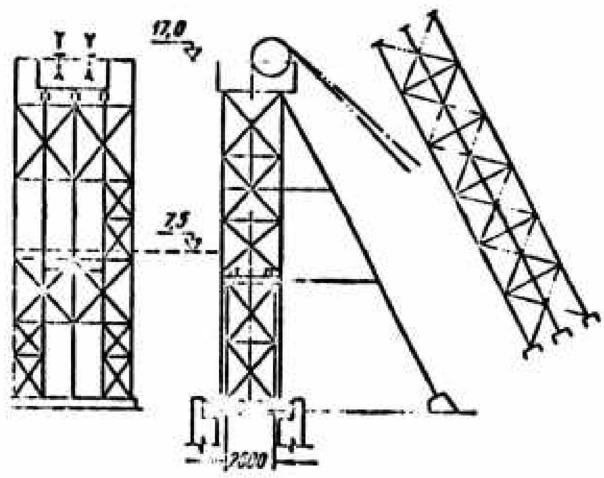

Fig. 149. The diagram of a wooden machine mounted pile driver having a height of 17 m.

All the wooden elements of the booms are protected on three sides, by painted roofing iron. The mount and hip of the head of the pile driver have continuous sheathing.

The average value of a coefficient *a* in the formula of the volume of wood in the body for machine mounted and diagonal pile drivers is equal to 0.4-0.6; the volume of wood in the body constitutes about one half of the corresponding volume in hipped pile drivers.

The described hipped, half-hipped and machine mounted wooden pile drivers are convenient for a haulage in a direction, parallel to the plane of lift. The haulage crosswise for these pile drivers is possible with the acquisition of a cage lift with a counterbalance. In all these cases the pulleys of the lifts are located on one under-pulley landing.

Haulage crosswise during double-cage lift with an arrangement of the pulleys in one vertical plane on two underpulley landings for all the enumerated wooden pile drivers is hampered and not used. An exception to this rule are the various complex designs of the head of pile drivers. Thus, for instance, with a hipped, half-hipped and machine mounted wooden pile drivers it is possible to place one pulley over the other pulley by means of introducing heads of special metallic coupled trusses with rigid lower chords (Fig. 150a) into the composition. The additional expenditure of metal structures using pulleys with diameters of 1.6-2.5 m constitutes 3-5 t.

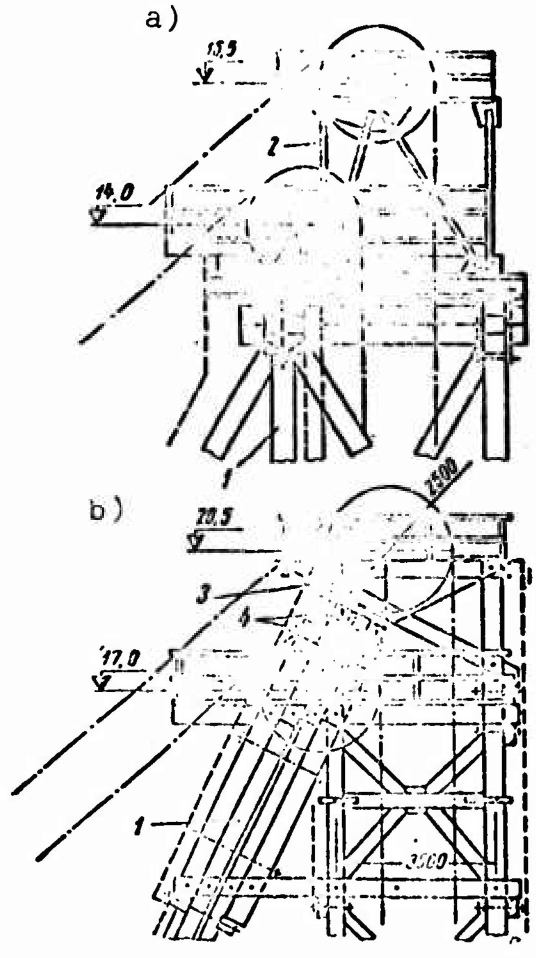

Fig. 150. The head of wooden pile drivers with an arrangement of the pulleys in one vertical plane: a) head of a hipped, half-hipped and machine mounted pile driver; b) the head of a diagonal pile driver; *1* – wooden frame of the construction; *2* – coupled metallic trusses with a rigid lower strap; *3* – underpulley trusses; *4* – transverse beams.

The wooden pile drivers of the diagonal system, to a greater extent, are suitable for the double-level arrangement of pulleys. Figure 150b depicts the head of a wooden diagonal pile driver during double-cage lift with haulage, perpendicular to the plane of lift. The pile driver having a height of 20.5 m carries two pulleys with a diameter of 2.5 m, arranged in one vertical plane on two variable underpulley landings at a level of 20.5 and 17 m.

The boom of a pile driver in its upper part has parallel straps; the width of the boom is somewhat less than the width of the machine mount. The short longitudinal rods of the boom directly sustain the loads from the lower pulley, transmitted through the grating of the boom to its straps. The rods of the straps of a boom extend higher than the lower of a lower underpulley landing to the level of the transverse metallic paired beam, which intersects the central plane of lift above the overall size of the lower pulley. The transverse beam is fastened to straps of the booms and to the machine mount. The two underpulley trusses set on the transverse beam are connected with one another outside the overall sizes of the upper pulley by broadside transverse beams, one of which is fastened to the machine mount in its external plane, which assures stability of the underpulley trusses with the random assembled loads of the upper underpulley landing.

For the described diagonal pile driver 58 m^3 wood is necessary in the body and 4.5 t of metal, including the weight of the structures of the head and joints (without the weight of the sub-pile driver beams). In this way, coefficient a in the formula of the expenditure of the wood of the pile drivers is equal, in this case, to 0.46. For a metallic machine mounted pile driver, under analogous conditions, about 40 t of metal structures would be required.

The joints of the pile drivers are characterized by the presence of considerable dynamic loads, developed landings of notches for the transmission of forces, and utilization of very simple notch-stops.

The latter is necessary for the reliable transmission of forces and for providing a normal quality of operation for the joints.

The chief loads of pile drivers – the loads of the lift, which are transmitted to the construction at the levels of the underpulley landing, are not constant, and continuously appear and are removed, and the application and the liquidation of loads are accompanied by jerks in various directions, by blows to the hauling vessels and so forth. With special (practically instantly appearing and just as rapidly disappearing) loads in the wooden pile drivers it is necessary to consider the unavoidably appearing dangerous reactive loads, frequently not paid attention to.

Allowing for the given design of the main joints of the construction one should calculate the possibility of temporary alternating forces in the joints. In the rods, adjacent to those joints, one ought to create a certain preliminary compression, which prevents the joints from opening and prevents the loss of form with operational loads. These measures pertain to joints at adjoining underpulley landings, to the unions of the boom with the body of the pile driver or a machine mount and to basic supporting joints, where one ought to provide anchor bolts.

The simplicity of the joints has a substantial value. The joints, presented in Fig. 151a, b, c, are examples of simple joints of hipped, half-hipped, machine mounted wooden pile drivers; in Fig. 151d and Fig. 150b – diagonal wooden pile drivers. The use of metallic boom somewhat simplifies the joints because of the more convenient bracing bolt connections to the metallic structures of the boom. General principles of the conservation of wooden designs should be applied in the solution of joints. Wood in the joints should be protected not only from the direct effect of atmospheric moisture, but also from condensational moisture on the metallic surfaces. To avoid the accumulation of moisture, the joints with the utilization of metal should not be enclosed, which also facilitates

good ventilation. In the planes, where the contact of the wood with the metal on a large surface is unavoidable, one ought to perforate the metal, and the wood is thoroughly protected by the padding of a ruberoid and sanitized. Near the foundations the wood should be sanitized to a height up to 1 m. Sanitation should also be applied in all necessary and doubtful cases.

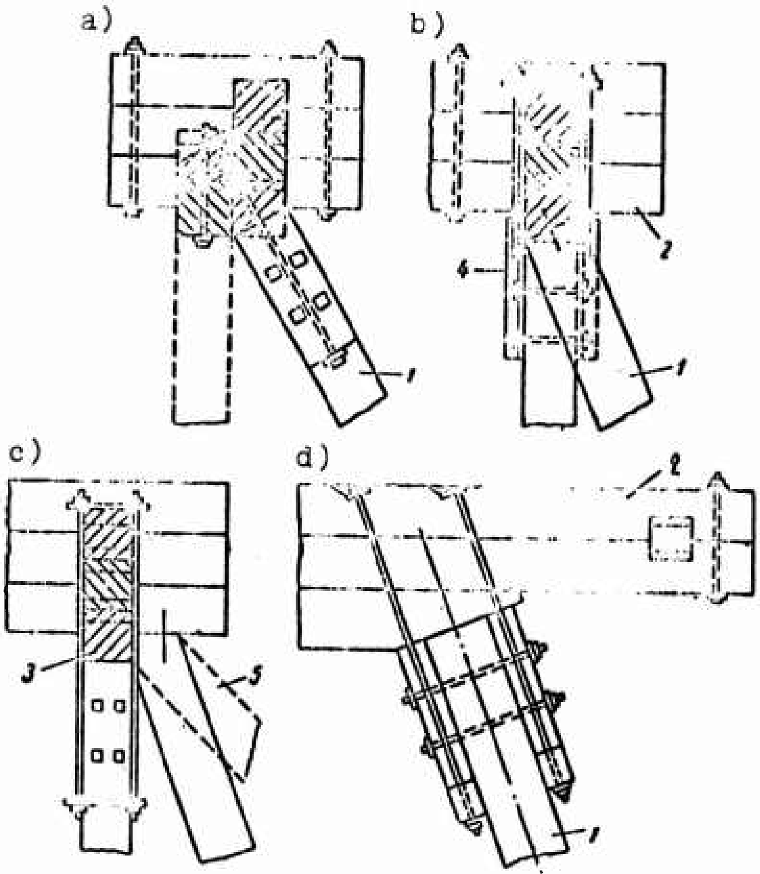

Fig. 151. The basic joints of wooden pile drivers: a, b) abutment of the boom to the rear trusses of the pile drivers; c) abutment of the boom to the rear trusses and braces to the front trusses of pile drivers; d) abutment of a boom to the underpulley beam; *1* – a boom; *2*, *3* – underpulley and transverse beams; *4* – tension equipment; *5* – bracing.

In all critical joints, where the accumulation of moisture is possible, and ventilation limited, especially for semi-enclosed gaskets, and abutments to metal and concrete in the supports and in other cases, the utilization of larch; a wood which is characterized by higher resistance to rotting and by high mechanical indexes is recommended. In remaining cases for the manufacture of rods of pile drivers one ought to apply selected pine wood.

Of great significance for the preservation of wood in a construction is the level of its arrangement. In all possible cases it is recommended to raise the concrete and ferroconcrete supports of the pile drivers to a height of not less than 0.7 m above the surface of planning. Only in extreme cases a reduction in this height to 0.5 m is possible. Under mine conditions one ought to consider the inevitable increase in the level of planning during operation. It is recommended to arrange space prop walls made of ferroconcrete slabs between the foundation posts.

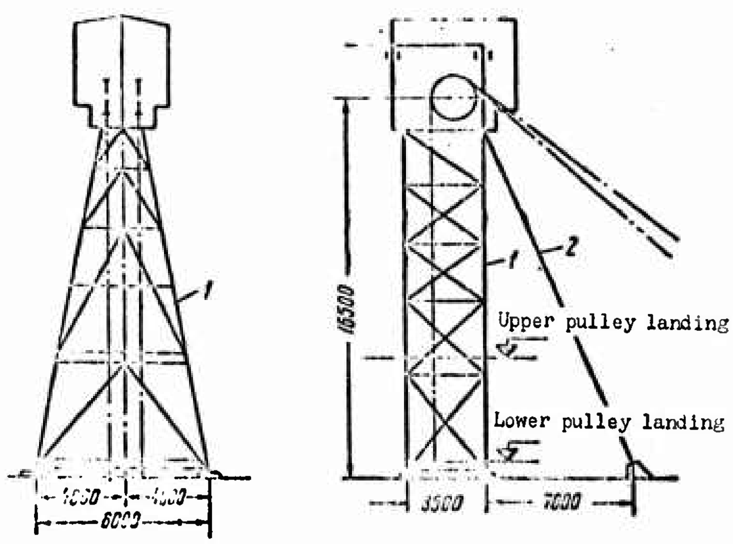

Fig. 152. The diagram of a wooden half-hipped pile driver with a metallic boom: *1* — wooden designs of a half-hipped pile driver; *2* — metallic boom made from two channel bars with a grating made from light-weight angles.

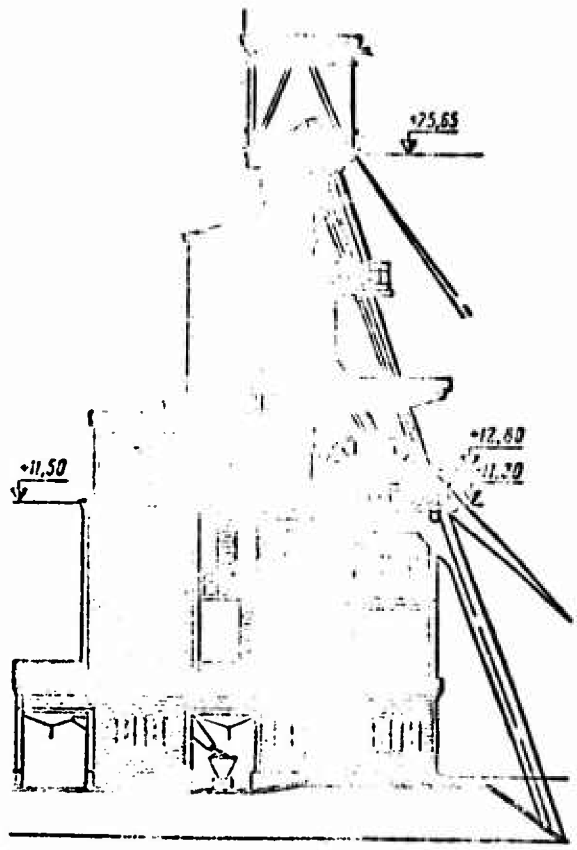

Fig. 153. The pile driver of a complex design using reinforced concrete in the lower part and wood in the upper part of the construction.

Various complex designs of pile drivers are known, based upon the utilization of wood and other material. Figure 152 gives a diagram of the pile driver of a half-hipped system with wooden body of a pile driver protected from moisture, and spread crosswise, and with a flat metallic boom, The boom, which possesses, in this case, parallel straps made from channel bars and the simplest grating from lightweight angles, is fastened to the concrete foundation and to the machine mount of the pile driver with bolts. The adjoining of the boom to the machine mount was accomplished by one of the simplest methods, shown in Fig. 151c, using a horizontal channel bar. This pile driver is used at one of the Ural mines for more than 20 years.

Figure 153 shows a wooden-reinforced concrete pile driver at an Altay mine. In this case the wooden pile driver, which has a height of about 14.3 m, is set on ferroconcrete supporting frames having a height of 11.3 m. In this way, the complete height of construction is equal to 25.6 m.

On the basis of the combination of wooden and other designs with cable steel guy wires and stretchers there can be a number of complex designs of the pile drivers which are described in the examination of metallic pile drivers.

6. Sinking Pile Drivers

Sinking pile drivers are a type of a deckhouse structure with guide pulleys and are examined here quite briefly. During the examination of sinking pile drivers one ought to have some information about the driving and sinking equipment.

Driving by the usual means amounts to the breaking of the rock predominantly with shot-firing work, the loading of the rock into lift vessels, the lift of the rock to the surface with its transport to the dump. Simultaneously with these activities a permanent, and sometimes temporary shoring of the stem of the mine shaft is erected drainage is provided with the aid of suspension sinking pumps or buckets and with the ventilation of the face. The ventilation ducts, compressed air lines, for the descent of the concrete mixture, cementation or plugging, the cables suspended by ropes; recently the suspension of pipes and cables has been done on the walls of the enclosed shoring of the mine shaft.

The lifting of rock, descent of material and structures is done in buckets having a capacity of 0.5-3 m^3. Cables spaced symmetrically on two sides of the bucket usually serve as guides for the lifting of the bucket. The guide frame of the bucket lift slides past the cable. The tension of the guide cables is produced using a winch set up at the surface, and the lower end of the cables are attached to the taut metallic frame, or the strained safety flange. The buckets are also used for the descent and lift of passengers.

The erection of a permanent shoring is done using a metallic suspended flange in which there are a number of openings for the passage of the suspended pumps, pipe lines, sinking buckets, rescue stairway, and so on.

The reinforcement of the mine shaft (laying the buntons, suspending the guide rails) is done using suspended flanges and cradles For the lifting of the above mentioned and other equipment, a number

of hoisting winches are used, the number of which reaches 30 or more for the driving operation. Sinking equipment is suspended from steel cables, guided by pulleys, set on the sinking pile driver whose load is transmitted to the sinking pile driver. Only recently has there been a shift of the loads from certain equipment to the enclosed reinforcement of the mine shaft. The driving of the shaft stem using a method without a pile driver merits special attention when the pulleys are not attached to the sinking pile driver, but to the construction in the mouth of the mine shaft stem.

Metallic and wooden sinking pile drivers should be regarded as stock-supplied, usable for driving certain mine shafts and maintainable in a ready state for driving routine shafts. Recently, metallic sinking pile drivers, most suitable as stock-supplied equipment, are predominatly used.

In distant and remote areas, with shallow shafts, small capacity sinking buckets can be used along with wooden sinking pile drivers. The driving with wooden pile drivers is also expedient in shallow stems of exploratory shafts when the pile driver is put to actual use following the driving.

In the enumerated cases the simplest schemes of setting up the sinking equipment with the minimum amount of mounted lift machines, winches and corresponding pulleys on the pile drivers, are used. Figure 154a shows a similar scheme of the layout of sinking equipment for a stem of a mine shaft, and Fig. 154b represents a scheme of a small wooden sinking pile driver 13.5 m high. The pile driver has a number of features. The planes of the pulleys are parallel to the planes of the three main trusses of the pile driver, which are located at eqiudistant from one another. Each main truss is composed of two supporting triangles, located a certain distance from the stem, and with a three-way swivel arch with metallic stay installed on them. The joints of this structure are formed by simple front cutting-stops (Fig. 155). The underpulley landing is represented by cribwork

beam cage with underpulley and transverse beams made from bars with rough edges, cut into quarters made of wood. Sanitized foundation beams or concrete pillars are used as foundations of a pile driver.

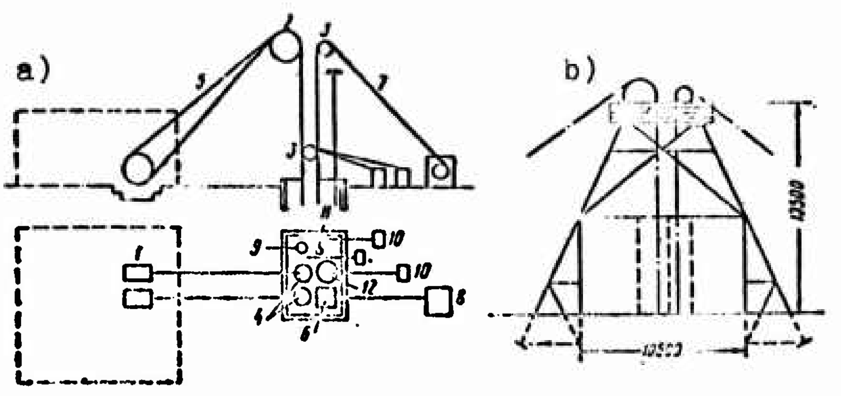

Fig. 154. The simplest diagram of the arrangement of sinking equipment for the stem of a mine shaft a) and a diagram of a small wooden sinking pile driver b); 1 - lift machine; 2, 3 - pulleys; 4 - buckets; 5, 7 - cables; 6 - sinking pump, suspended; 8, 10 - winch with mechanical and manual drive; 9, 11 - pipelines; 12 - pneumatic loader.

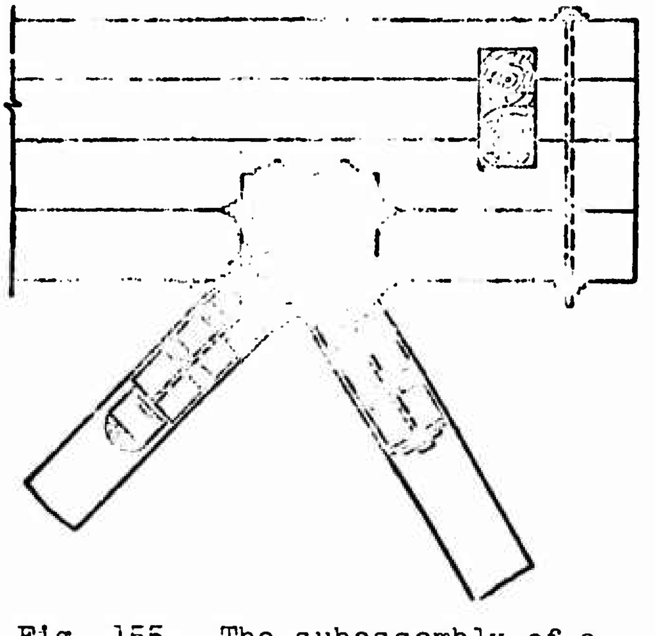

Fig. 155. The subassembly of a wooden sinking pile driver.

The total number of pulleys, installed on such a wooden sinking pile driver, usually does not exceed 8-12. The designs of the described type of pile drivers differs in simplicity and reliability, and they have an allowance for a certain overload and are repeatedly used with good results for driving shallow stems of mine shafts in the Urals region.

Needed for the manufacture of a pile driver with an underpulley landing a machine mount and planking are 50 to 70 m^3 of wood and 2-3 t of metallic items (belts, bolts, cover plates, washers).

Lightweight metallic sinking pile drivers used under analogous conditions usually have the outlines of a frustum with a height of about 14 m with a square base 8-10 m on the side. These pile drivers are characterized by the presence of diagonal connections in the basic planes, and by the use of pipes for bars, struts and crosspieces. The number of pulleys on the underpulley landing usually does not exceed 10-12. The weight of the lightweight connections of metallic tubular pile drivers with flanged main unions, used for the simplest schemes for the arrangement of sinking equipment mentioned here, constitutes 15-20 t.

In the copper ore industry metallic frame sinking pile drivers are used. Main frames of these pile drivers are characterized by their comparatively large span at the base, which attain 15-20 m. At heights of 10-14 m above the planning surface crosspiece of the frame is placed. The span of the crosspiece because of the slope of the uprights of the pile driver frame is equal to 5-8 m, the height of a pile driver is 16-20 m. The crosspiece and the inclined uprights of the main frame of the pile driver - welded in a continuous double-T section. For less heights the uprights are composed by rolled double-T No. 55a beams with sheets; with a height of a pile driver at 20 m the section of the uprights - variable with the height of the wall of the double-T (made from sheets, 10 mm thick) - towards the top (at the level of the crosspiece) 1000, towards the bottom (at the level of the foundation) 500 mm with straps, 250 × 25 mm.

The main frames are set at a distance of 4.5 m. During the driving of the stems 7-8 m in diameter the pile driver is supplemented by two auxiliary frames. All the frames are united by grated connections from the corners. The underpulley landing of the frame pile driver is characterized by the presence of continuous flooring made of beams. Above the flooring sinking pulleys are installed on the auxiliary structures. The sinking winches and lift machines are, as a rule, set parallel to the planes of the frames of the pile drivers, in connection with the fact that the possibility of the pulley device on the underpulley landing is somewhat limited, but the number of additional deflecting (stop) pulleys is relatively increased.

The basic type of sinking pile drivers are the metallic connecting tubular stock-supplied sinking pile drivers, is widely used in the coal and ore industry (Fig. 156). The pile drivers serve for the driving of stems at a depth of 200-1000 m, with a diameter up to 8 m. Pile drivers of the assemble-disassemble type can also be used on several headings. The assembling of metallic structures is don on rough bolts. Most frequently four types of pile drivers are used for driving the stems to a depth to 800 m; and with a diameter up to 7.5 m (Fig. 157); their characteristics are give in Table 28.

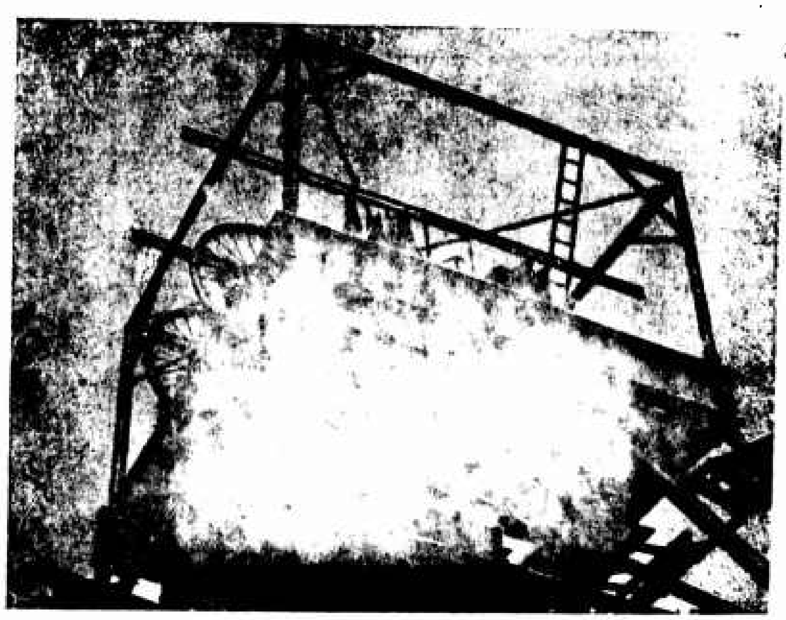

Fig. 156. The upper part and underpulley landing of stock-suppled metallic sinking pile driver.

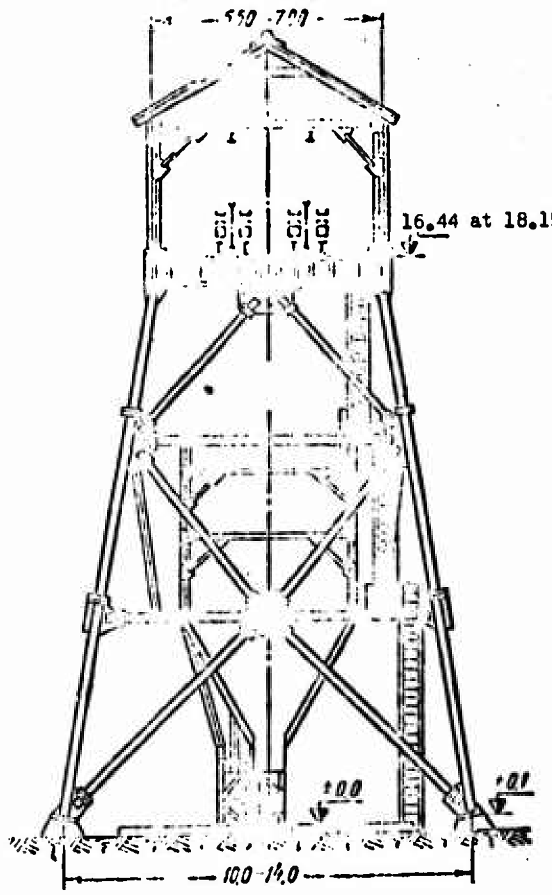

Fig. 157. Diagram of a stock-supplied, assemble-disassemble metallic sinking pile driver.

Table 28.

Designation of the indexes	Type of pile driver			
	I	II	III	IV
Depth of the stem, m.	200	400	600	[illegible]00
Diameter of the stem, m.	4,5—6,0	5—6,5	5,5—6,5	6—7,5
Span of uprights at base of pile driver, m	10×10	12×12	12×12	14×14
Sizes of underpulley landing in plane, m.	5,5×5,5	6×6	6,5×6,5	7×7
Height from top of foundation to the top of underpulley landing, m.	16,34	17,35	17,[illegible]5	18,05
Weight of metallic designs of the hip of the pile driver, t.	19,5	23,5	26,1	[illegible]3,6
Weight of main beams of the underpulley landing, t.	4,5	5,9	8,2	11,0
Weight of the stairs, t.	1,2	1,3	1,2	1,3
Weight of metallic frame of pile driver (without the weight of the auxiliary beams of the underpulley landing), m..	25,2	30,7	34,5	40,9

Table 28 gives data on only the weight of the main beams of the underpulley landing, which are located on the landing and at its middle. For the first two types of pile drivers the average beams - single, for the pile drivers of type III and IV - double. In this

way, the distance between the main beams of the landing does not exceed 3.3 m, which allows having comparatively small sections of auxiliary (underpulley) beams.

The arrangement of the auxiliary beams of the underpulley landing is taken in each separate case, depending on the arrangement of the sinking equipment in the stem and the arrangement in the sinking lift machines and winches at the surface. The sections of beams are taken in accordance with the loads on sinking pile drivers mentioned below. The weight of the metal structures of secondary beams in a number of cases is found to be within the limits of 4-10 t. The designs of the unloading landing also are taken in accordance with the local conditions of driving, which corresponds to the expenditure of the metallic designs approximately equal to 5-8 t.

The total weight of the metallic structures of the sinking pile drivers of types I and II, constitutes on the average of about 35-40 t, but pile drivers of types III and IV, 50-55 t.

For the driving of the stems to a depth of up to 1000 m, with a diameter up to 8 m inclusively, pile drivers of type V with a height of 18.6 m along with a spread of the uprights at the base of the pile driver, 14 × 14 m, and an underpulley landing of the size 7 × 7 m, in a plane, are used. The total weight of this sinking pile driver constitutes 60-65 t.

Figure 158 gives a diagram of the arrangement of the sinking construction of the stem of an ore mine shaft. As a rule the lift machines of the bucket lift devices are set along the large axis of the landing of the sinking construction. In most cases all the sinking lift machines and winches fall in the plane of a rectangle with sides, 80-120 × 40-80, and as a first approximation, 100 × 60 m.

Loads on the underpulley landings of the sinking pile drivers are set by a scheme of driving and they are determined by the weight of the sinking equipment (Table 29).

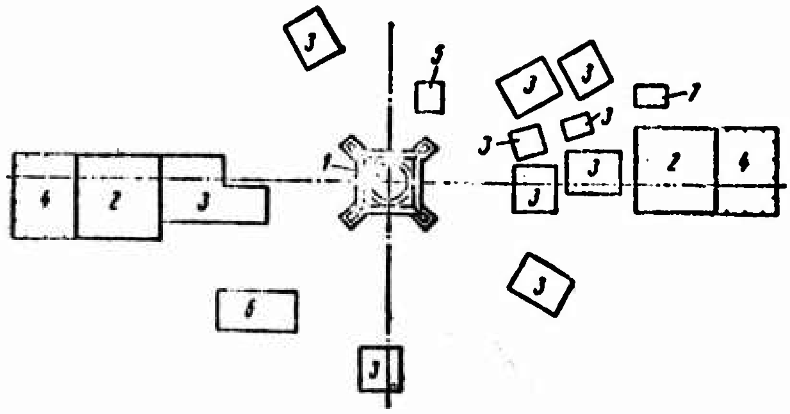

Fig. 158. The diagram of the arrangement of the sinking construction for a vertical stem of an ore mine shaft: 1 - stock-supplied sinking pile driver; 2 - housing of the lift machines; 3 - groups of sinking winches; 4 - temporary electric substation; 5 - ventilator; 6 - mobile compressor; 7 - mobile power station.

Table 29.

Designation of the equipment	Weight, t
Muck bucket, with a capacity of 1 m^3	2,8
The same 1.5 "	3,8
" 2 "	5,2
" 2.5 "	6,2
Bucket with a capacity of 1 m^3	3,0
The same 1.5 "	4,0
PPN-50s suspension pump	3,4
VP-2 high-pressure suspension pump	4,0
Drainage pipe of a diameter, 150 mm with water at 1 m	0,05
Ventilation pipe of a diameter of 400 mm at 1 m	0,04
The same, with diameter of 800 mm at 1 m	0,07
Compressed air pipe with a diameter of 100 mm, at 1 m	0,02
The same with diamter of 200 mm at 1 m	0,04
Cementation pipe with a diameter of 50 mm at 1 m	0,02
Rescue stairs with the diameter of stem up to 5.5 m	1,9
The same 6.5 "	2,3
" 8 "	2,9

The weight of the sinking bucket in Table 29 is presented allowing for the weight of the housing of the bucket, trailer equipment, guide frames, rock, or material and pulp. The weight of the pumps is given allowing for the weight of the worker with his tools. The weight of the sinking flange with the diameter of the stems at 4.5, 6, 7, 8 m, respectively, constitutes 7, 10, 14, 18 t. The total weight of the loaded flange depends on the design of the shoring and organization of the work during the driving of the stem. For the diameters of the stems enumerated here the weight of the loaded flange can comprise 15, 18, 25, 30 t. Included here is the weight of the

containers, the instrument, concrete blocks or brick, mortar, concrete, quarrystone, winches with pneumatic loaders and the weight of the workers, on the flange.

The value of the stretching cable guides is assumed to be equal to the load-lifting capacity of the appropriate winches. With the depth of the stems up to 200, 500, 800 m winches with a load-lifting capacity of 3, 5, 10 t respectively are installed.

During the determination of the forces in the cables one ought to consider the means of suspending the sinking equipment. Buckets, sinking cages, rescue stairs are usually suspended from one cable. The suspension pumps and the fixtures of drainage, ventilation, compressed air and cementation pipes are suspended on two cables. The sinking flanges are suspended from one, two and four cables.

The normative load of the cable of a bucket or a pump is determined, for example, by the sum of the weight or half-weight of the equipment and the weight of the cable. The sum total breaking force of the strands of the cable should exceed the given load by 5-9 times. Specifically, during the lift of a loaded bucket the margin of safety should not be less than 7.5, but during the lift of the rescue stairs and the bucket with passengers - not less than 9.0. The safety factors for cables of the drainage pipes, pumps and sinking flanges should not be less than 6, but for cables of ventilation, compressed air, and cementation pipes, guide cables, which - not less than 5.

During the calculation of cables the coefficients of overload do not have a value. In the calculation of designs of sinking pile drivers the coefficients of overload for temporary loads can be assumed within the limits of 1.3-1.4, pertaining to large values for work loads of flanges, rescue stairs, buckets. The coefficients of dynamicity, connected with the work of the bucket, in particular, is not considered. The coefficients of overload as applied to the intrinsic weight of the bucket, flanges are equal to 1.1-1.2. Considering the difficulties, connected with the separation of the

numerous loads from the equipment, it is possible to accept an averaged coefficient of overload for the loads of the majority of sinking equipment, equal to 1.3. The obtained calculated loads in this instance, specifically for loads of a sinking flange, can be specified.

The coefficient of overload for the intrinsic weight of the designs of the pile drivers (frames, auxiliary beams, planking), the weight of the cables and pulleys is equal to 1.1.

In individual less clear and unstudied cases this coefficient is assumed to be equal to 1.2. The coefficients of overload for windy and snow loads in accordance with the data of the Construction Standards and Rules, are equal to 1.2 and 1.4. A temporary and evenly distributed load on the underpulley landings constitutes 100 kg/m^2 with a coefficient of overload of 1.3

The effect of the working loads of cables, the weight of the pulleys and weight of the designs (allowing for other usually appearing loads) is considered as the basic combination of loads. The same goes for the effect of the wind as an additional combination. The effect of the rupture of the cable of one bucket during a double hoisting load in the cable of the conjugate bucket with the same lift and working loads in all the remaining cables, allowing for the weight of the pulleys and structures as well as the wind, is considered to be a special combination of loads of a sinking pile driver. One ought to consider cases of rupture of each of the cables of the bucket lifts. The rupture of the cable of these lifts is possible in connection with considerable velocities of motion of the vessels. The rates of motion of the remaining sinking equipment are small; therefore, cases of rupture of the corresponding cables under usual conditions do not take place and they are not taken into account.

The coefficient of overload of the rupture of a cable should be taken equal to 1.4. Allowing for this special load the value of a calculated temporary load accordingly to the Construction Standards

and Rules is multiplied by the coefficient 0.8. The given value of the coefficient of overload with the rupture of the cable is the least permissible value.

7. The Prefabrication of Pile Drivers

The complexity of prefabrication of pile drivers involves not only difficulties connected with the development of these or some other designs but also the complexity in the problem, the need for the simultaneous tying in of the problems of the lift, the over-all size of the prefabricated vessels, their entry into the pile dr[illegible] haulage and unloading with prefabrication as the solutions of the [illegible]r.

The labor input of prefabrication is determined by a considerable number of combinations of schemes of pile drivers, which are governed by many factors. Nevertheless, the prefabrication of pile drivers is possible and it should be developed. Effort in this direction has actually just started in the ore industry.

In the coal industry prefabrication exists in a part of standard metallic pile driver machines. Prefabrication has encompassed skip lifts with skips of 3, 4, 6 and 9 t, and lifts with overturning cages for two-ton and three-ton trolleys. The height of the skip pile drivers is 31, 34, 43, 46 m, the cage - 29.5 m.

The levels of the receiving funnels of the skip pile drivers at heights of 31, 34, 43 and 46 m constitute 17, 20, 29, 32 m, respectively. The diameters of the guide pulleys is accepted as equal to 3, 4 and 5 m.

One ought to note that the calculations of standardized pile drivers are conducted for two diagrams of skip pile drivers having heights of 34 and 46 m, and for one diagram of a cage pile driver, i.e., for all three diagrams, with their overall number, 27.

For the calculated diagram of a skip pile driver having a height of 34 m skips with a load-carrying capacity of 6 and 9 t, pulleys of 4 and 5 m with a breaking force of the cables of 96.8, 113.0, 161.5 t, distance to the lift machine from the axis of the stem, 29.5-52.5 m are accepted.

The diagrams of skip pile drivers having a height of 34 and 46 m are given in Fig. 118; the diagram of the cage pile driver is given in Fig. 119.

When prefabricating mine pile drivers, one should primarily consider the single-hoisting mounted and diagonal pile drivers with plane and plane improved booms in a lower part.

At a considerable height of the construction it is expedient to consider the same flat mounted or diagonal pile drivers with their specified stretching cables, directed outside the plane of lift. The designs of these pile drivers at least are subject to change with the increase in the height. With an increase in the height of the pile driver within certain limits, for example, 3 and 6 m extra panel inserts can be added to boom and frame of the mount of the diagonal pile driver, but stretching cables are correspondingly somewhat lengthened. This can be augmented with the retention of sections of the elements of the boom and cables and the invariability of the designs of the abutting joints of the cables to the foundations and to the pile driver. Approximately the same volume will correspond to a certain increase in the height of the mounted flat pile driver with stretching cables and so forth. The earlier enumerated pile drivers of type ST-1, ST-3, R-1, R-3 to a large degree are suitable for pruposes of prefabrication.

The prefabrication of frame pile drivers is comparatively more easily attained because of the small quantity of various joints and the absence of bracings of the grating. It is obvious that the indicated advantage will be most successfully accomplished using a combination of frame design with the most effective system in relation to typification.

In this way, better results in prefabrication of small frame pile drivers with moderate loads should be expected as a result of utilizing the following designs:

ST-I-P - mounted rigid frame pile drivers with unimproved booms from the plane of lift;

R-I-P - diagonal rigid frame pile drivers.

With the increase in the loads and height of the structure it is expedient for the purpose of prefabrication to subsequently consider the utilization of the following mounted and diagonal pile drivers: flat grated pile drivers; rigid grated pile drivers with a flat boom, spread out crosswise in a lower part; flat grated pile drivers with stretching cables, directed from the plane of the lift.

During the examination of the problems of the prefabrication of ferroconcrete and reinforced concrete pile drivers, one should primarily consider the bracing-upright flat and flat sectional reinforced concrete pile drivers with stretching cables.

8. The Manufacturing and Operating Procedure of Mine Pile Drivers

Deckhouse pile drivers are the most critical structures at the mine. The normal operation of the pile drivers is associated with the lift and the descent of passengers; furthermore, pile drivers serve to lift the ore, spoil and various materials. In connection with this, there are high requirements set for the manufacture of pile drivers.

Inasmuch as welded metallic pile drivers of significant sizes are predominantly used in mines, which frequently handle several lifts and which receive large loads in the process of operation, the control for the quality of the manufacture and the current state of the steel designs of the pile drivers, manufactured using welding has special value.

The welded structures of pile drivers are made from carbon steel of conventional and high quality, from carbon steel for bridge building and, partly from low-alloy structural steel. The carbon steel, and also the low-alloy steels, used for the manufacture of steel structures, should be adequate for the requirements of the Construction Standards and Technical Conditions. Since pile drivers are actually used even at low temperatures and sustain various dynamic loads, then the steel should satisfy the requirements of impact toughness at a negative temperature.

Among the group of carbon steels meeting the requirements for steel structures of pile drivers, there are steels M16S and VSt.3 (GOST 380-60 and GOST 6713-53).

Carbon hot-rolled steel for the welded bridge structures of brand M16S (GOST 6713-53) is recommended for use in making the elements of the structures, which directly sustain dynamic loads (underpulley trusses and beams, transverse trusses and beams, and other separate elements of the head of the pile drivers; underside stop beams and trusses; structures, sustaining the loads of rolling stock, the blows of vessels) and even used at a temperature -40° and lower.

For the manufacture of the listed steel structures of pile drivers at a computed temperature -30 to 39°, it is possible to use structural, martensitic steel of brand VSt.3, equipped with additional guarantees in relationship to impact toughness.

One can recall that with the delivery of steel under group B, additional guarantees are necessary concerning the mechanical properties and, specifically, the tensile strength, the yield point and relative elongation. Satisfactory indexes based on the impact toughness of the steel is assumed only when meeting the corresponding specified requirement.

In the case of the utilization of steel, falling into group B, additional guarantees of the mechanical features of the steel,

specifically the tensile strength, the yield point, relative elongation, impact toughness are necessary. In this way, under operating conditions, for example, instead of VSt.3 steel, brand MSt.3 is used (martensitic steel of group B is the same under GOST), the tensile strength should constitute 38-47 kg/mm^2, the yield point for the first grade of rolled steel not less than 24 kg/mm^2, the relative elongation of a long sample of not less than 21-23%, impact toughness at a normal temperature of not less than 10 kg-m/cm^2 for sorted and shaped steel and 8 kg-m/cm^2 for sheet and broadband steel. The shown values are established for the corresponding tests on the mechanical qualities of the steel.

Individual enterprises, not specialized in the manufacture of steel structures, but manufacture steel designs and produce different steels should without fail when using welded structures make the corresponding tests on the steels they produce. The nonfulfillment of these requirements can lead to mass waste and damage.

During any replacement or introduction of a new brand of steel it is necessary to keep in mind that the overall deficiencies of old and new GOST for carbon steels are lowered requirements (as opposed to Construction Standards and Rules and practical data) for the value of impact toughness. In most cases the requirements of the GOST are limited to the data on impact toughness at a normal temperature.

In a number of cases the value of impact toughness of steel should be determined at a temperature of -40° and to constitute, in this case, 3 kg/cm^2 and higher. For steel in the designs of the construction, actually used at a temperature of -30° and below and in support structures the mobile loads (in the structures of mine pile drivers, deckhouse building, haulage galleries, hoppers and others), such tests are necessary.

The utilization of structural low-alloy steel of brand 15KhSND(NL-2) and nonnickel steel of brand 14G2, 10G2S, 15GS results

in considerable economy of metal. Steel 10G2S is characterized by temporary resistance to a tension of 50 kg/mm^2, by yield point of 35 kg/mm^2, by a strength under vibratory loads greater than the strength of steel of brand 15KhSND; the value of impact toughness at a temperature of -40° - is more than 3 kg-m/cm^2. Structures made from steel of brand 10G2S are characterized by a reduction in their weight in comparison with structures made from steel St.3kip. by approximately 15% without an increase in the cost of the construction.

Today the most widely used is low-alloy structural steel of brand 15KhSND(NL-2), which is characterized by a tensile strength of 52 kg/mm^2 by a yield point of 35 kg/cm^2, is practically ready-made for the manufacture of a number of structural designs. Wide use of this steel is not always economic profitable. In a number of cases, however, it is feasible and expedient for the manufacture of small rods and joints of mine construction, supporting large loads, made from steel 15KhSND, which can result in a reduction of the sizes of designs and in a decrease in the weight of the parts and joints by 15-20% in comparison with the weight of designs from steel of brand St.3.

Also, the utilization of other low-alloy structural steels has not been excluded from small parts of pile drivers and other structures.

A rather frequent error when developing the schemes for pile drivers is the introduction of insufficiently rigid, uncalculated elements, designed constructively because of the absence of any considerable forces in the rods. It is necessary to consider the transport and installation of the construction and to design rather rigid sections of the uncalculated elements of the machine mount, bracing, head and of separate rods, under the indicated conditions.

Sometimes proper attention is not given to clearances between the rims of pulleys and the structures of the underpulley landings, between the structures of the underpulley landing and the cables, between the elements of machine mount and the lift vessels,

specifically between the catching of the parachute devices and the buntons.

It is necessary to turn ones attention to the invariability of the machine mount, which is relative to the extended mine shaft. For the shown purpose, rigid and diaphragms, and furthermore, the elements of rigidity in the joints of the machine mount, facilitating the invariability of the latter, are necessary. With large sizes of such a mine shaft and with considerable length in a number of cases it is necessary to insert rigid straps, located along the external outline of the machine mount. The least measures are necessary for this to a greater degree since the lighter elements are accepted for the manufacture of the machine mount.

The volume of the graphically designed material is determined by conventional planning of the metal-design phase, [KM] (КМ), whereby it is recommended to include in the composition the general forms of a pile driver and designs of the head of the scheme of the lift, specifications and basic requirements for the manufacture of the designs. The metal should, in order, show the brand of steel and its delivery for the manufacture of the welded structures. In all cases it is necessary to specify the state standard of the guarantee, for example, the value of impact toughness at a temperature of -40° for carbon steel and others.

During the manufacture of the steel structures of pile drivers errors are observed rather frequently in the marking of the opening, their burning completely through, the absence of a correction of a number of elements, the utilization of available sheet and other unmarked steel, sheet steel of high carbon content, the utilization of unrated electrodes and low quality of welding, specifically, the absence of developed weld techniques and a disorderly placement of the seams.

The shown defects usually are formed during the making of the steel designs by forces of nonspecialized enterprises under conditions

of the production operations by economic methods in the regions of individual mines. Under such conditions continuous control is necessary for quality of the manufactured product and the installation of the steel structures of the mine construction. This control should be conducted, beginning with the problems of laying out the construction and dividing it into shipping units and ending with the questions of the acceptance of a pile driver in operation.

In accordance with the noted and other questions under technical conditions in the manufacture and installation of the steel pile drivers, it is necessary to provide the following.

The manufacture of the designs should be produced in accordance with existing instructions on the manufacture of steel designs made from carbon steel, on the manufacture of steel designs made from low-alloy steel and by other indications.

A pile driver should be broken down into individual units, convenient for transport shipping.

It is necessary to thoroughly break down the geometrical schemes of the axes of the trusses of the construction into actually measured sizes and to compare them with the plan.

The marking on the steel is made from patterns, taken from the broken down actually measured geometrical scheme.

The accuracy of laying out, marking and erection of the structures by actual measurement should secure the minimum deviations in the dimensions.

Permissible deviations (allowance)

Spans and lengths of the trusses of a machine mount and boom....................	±10 mm
The difference in the lengths of the panels within one brand....................	±2 mm

The length of the joints and trusses of a heading	±2 mm
The sizes of the sheets in the joints	+2 mm
The sizes of the guide curves	±2 mm
The sag of a rod of one brand from its length	1:1200
The tangent of the angle of rotation of the flanges of tubular pile drivers	1:1500
Clearances between adjacent flanges at a distance of 20 mm from the pipe	0.5 mm
Clearances for the external edges of the flanges	1.5 mm
Distances between the centers of the foundations of not more than	20 mm
The difference in the marking of the shoes and the displacement of the uprights	±5 mm
The sag of the straps of one brand after installation based on the length	1:1000
The deviation along the vertical axes from the designed position after installation of the construction, expressed in fractions of the height	1:1500

Negative tolerances with mount tension rods are taken according to the data in the plan.

All elements of the pile driver based on form, sizes and material, should correspond to the working drafts.

The sizes of the sheets are planned allowing for transvérse shrinkage of the welded seams of 1 mm per seam, and 2 mm per seam of a thickness of 10 mm and more. In the absence of the necessary profiles the uniform-strength elements and junctions of elements are allowed, arranged, arranged according to the indications in the plan.

Before marking the steel should be straightened. Considerably bent metal should be straightened in the state of a light yellow incandescence with inadmissable nardening and overburning. Correction in the state of dark-blue incandescence is not allowed; cold correction can be done on angle-straightening and sheet-straightening levellers or by using screw clamps. Heading, the bending of sheets, is done only in a hot state with subsequent slow cooling. After heading and other operations, the elements should not have cracks,

rents and other defects. Straightening and bending of structural steel should only be done under special instructions.

Steel for weldable elements can be cut off with shears, saw, autogenous welding; electric-powered cutting is not allowed. The surface after trimming is cleaned of slag, dirt and accretions of metal. The accuracy of cutting by autogenous welding at a thickness up to 20 mm and with automatic machines – 1 mm, and by hand cutting – 2 mm, for sheets predominantly using positive tolerance. The plane of cut should be normal to the surface of the sheet.

In the remaining cases trimming is done with shears with the provision of perpendicularity of the cut. Clearances in the joints and the type of sheathing are taken according to the data of the plan. The ends of the rods in the supports should be cut off especially accurately and should fit tightly along the entire area of the section. In separate cases specified by the plan the planing or milling of the ends of the elements is done.

The punching of holes is allowed for a thickness of sheets up to 20 mm with subsequent correction and complete boring through the entire stack up to the necessary diameter. In remaining cases the holes are bored to a full diameter according to the tolerances for finished bolts. The axes of the openings should be normal to the plane of the elements. The mutual displacement of the axes of the openings in the stacks of parts should not be more than 1:200 of the thickness of the stack. The bolt and rivet connections and the stretching cable mounts are made according to the specifications of the plan.

Riveting should be done with the pressing and central position of the head with the correct form provided to them. There should be no cracks on the heads of the rivet; notching and calking them is not allowed.

The filling of the openings with bolts and the abutting of the

nuts should be dense. The bolts in the structures with dynamic loads should be installed with two nuts. The main joints should be installed using selected rough, gauged or finished bolts. In the remaining cases rough bolts are used. The number of bolts or rivets in the joints should correspond to the plan. More than two washers should not be set under the nuts and heads of the bolts. The manufacture and testing of the stretching cables must be done in accordance with the specifications of the plan, but the sealing off of the cable ends in the bushings - according to the specifications. The mount tension cables are designed in the plan also allowing for corrections with a difference in the temperatures.

The welding of the steel structures should be done according to a worked-out technological process. The construction chart of the welding of the various joints must be drawn up taking into account the chart, accepted by the leading plants - the producers of steel structures.

In the construction chart specifications should be given on the welding of structures at positive and negative temperatures, on the heating of designs, electrodes, operating conditions and the sequence of welding, type of seams, and so on.

The welding is done, as a rule, at the lower position of the seams. The ceiling seams can allow only for uncalculated elements, hardly sustaining loads. Within the specified limits, the seams should be continuous and uniform in section. The length of the seams should not exceed that designed by more than 5-10%. Transitions from the section of a seam to the adjacent sections and from the seam to the basic metal should be even. Sharp changes in the sections of the seams, their porosity, accumulations, gas bubbles, unwelded craters, slag inclusion in the seams, cracks, ruptures of the seams, are inadmissible. Also inadmissible are any cracks in the weldable eleemnts and parts, cuts and other damage.

The hand welding of brand St.3 steel on underpulley trusses, underside stop beams and other structures with dynamic loads is done

with type E-42A electrodes, for the welding of the remaining structures electrodes of type E-42A and E-42 are used. The welding of low-alloy steel is done using electrodes of type E-50A and E-55A. With automatic and automatic-manual welding electrode wire is used under the layer of flux in accordance with the technical conditions and specification chart.

Only highly skilled graduated welders should be assigned to weld steel structures of pile drivers and analogous steel structures of mine construction.

A control assemblage of the pile driver is required for plant-manufactured steel structures. A completely assembled pile driver should be checked by technical control. The acceptance is made beginning from the marking and tool check of the geometrical schemes of the pile driver and ending with a check of the welding and other connections in all joints of the construction. All the noted defects should be removed. The parts ready for shipment are smeared with iron minium on drying oil; the seams, junctions, and cavities are filled with minium on drying oil using chalk.

When shipping the steel structures it is also necessary to ship all the planks, upright sheathing, connecting parts, small parts, and also the clamps for the scaffolds and the lift, stairs and others. All elements should be protected against bending and other deformations at transport and unloading.

The installation of the pile driver should be done according to plan. During installation one should regularly check the position of the axes of the designs. Final measurements with an indication of the deviations from the plan, produced after installation and alignment of the pile driver, should be worked into the schemes. After installation the structure is coated with two layers of oil paint.

The finished and ready-to-operate pile driver should have a name plate with the enclosure of basic documents, including the

building plan and working drafts, data on the manufacture of the structures, the delivery certificate and acceptance of the structure, data of mine-surveyor observations, made during the periods of laying out and installing the structure and during the delivery of the pile driver into operation. Among all these data one should turn his attention to the tying in of construction with the scheme of a lift, to the position of lift devices, underpulley and receiving landings, to clearances between the structures of the pile driver and lift vessels. Among the enumerated documents which should be applicable are "Books for the recording the results of an inspection of a lift device."

C H A P T E R IX

THE CONSTRUCTION OF HAULAGE

1. Haulage Trestles

Haulage trestles serve to transfer the rolling stock from a deckhouse building to the necessary height at loading hoppers, ore bins, the receiving funnels of concentrating plants, dumps, etc. Sometimes, the trestles connect the construction of two adjacent stems of a mine shaft to one another.

The trestles are usually characterized by height within the limits of 3-6, 9-12 and 15-18 m. The latter corresponds to the transport of the ore and waste race directly to the loading bunkers. The trestles of low height are frequently represented by designs with complete or partial utilization of wood. The higher and capital construction are usually made in the form of sheltered galleries.

In the overwhelming majority of cases the span structures of the trestles are horizontal or close to horizontal.

The trestles differ to a considerable extent, reaching sometimes several hundred of meters. The spans constitute 3, 4, 6 and 12 m and they attain 24-30 m and more. In the latter case for the

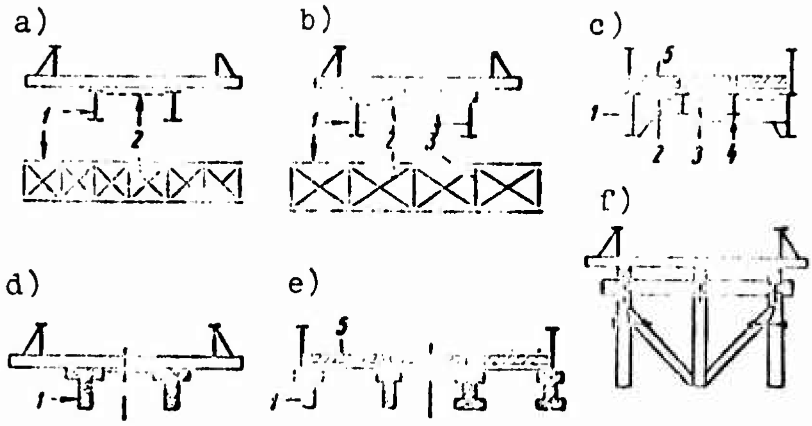

Fig. 224. The diagrams of the span structures of haulage trestles: a) with two main steel beams and ties, lying directly on the beams; b) mixed with main and transverse beams and with longitudinal auxiliary beams; c) with metallic beam cages and sectional ferroconcrete slabs; d) with two ferroconcrete or prestressed beams; e) diagram of a ballast span structure with ferroconcrete and prestressed sectional beams and sectional slabs; f) diagram of a wooden trestle; 1 - main beam; 2 - connections; 3 - transverse beam; 4 - longitudinal beam; 5 - sectional slabs.

In Fig. 224c - all-metal span structures from the main beams 1, connections 2, transverse 3 and longitudinal beams 4. With the introduction of slabs along with spans 3-6, and sometimes 12 m, the slabs are conveniently placed directly on the transverse beams 3. In this instance the longitudinal beams 4 are excluded.

Figure 224d and e show the analogous designs with the utilization of sectional ferroconcrete main beams. In one case the ties rest on the beams, in another case - sectional slabs. With the placement of the slabs on the crosspiece of the frames, the main beams 1 are excluded.

In Fig. 224f - the wooden trestle, in which the ties lie directly on the beams of a span structure.

Figure 225 shows the simplest haulage trestle with the

utilization of wooden designs. In Fig. 226 - the trestle, the span structure which corresponds to the diagram in Fig. 224b. Here, each span of a trestle has two main beams, connected to the semi-spans with transverse beams and, furthermore, uncoupled with these beams (in the planes of the upper straps) and with horizontal connections. From considerations of the economy of metal, wooden longitudinal beams are used. The main and transverse beams consist of double-T No. 45 beams.

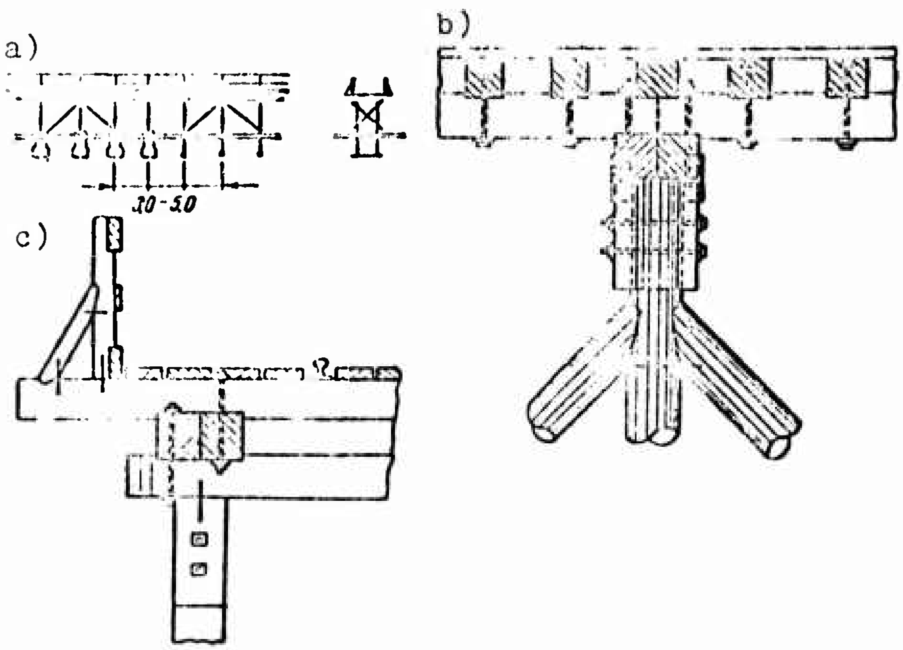

Fig. 225. Haulage trestle with wooden supporting designs: a) diagram; b, c) details.

On a doubletrack trestle having a width of 5.2 m, a height of 10.5 m and a span of 8.0 m, 0.7 t of steel designs per 1 m of length of construction is spent.

From the details of the span structure it is possible to describe the junction plates of longitudinal beams. One of the two braces located under each rail in each span, lies on the transverse beam and it projects beyond the face of the latter. The second

support structures of trestles, sometimes steel structures are used. The spans of the main beams and trusses of the trestles increase with the increase in the height of the construction and with the complexity of the foundations. The width of the trestles is equal to 3-4.5 and 6-9 m and it depends on the number of the rail tracks, set on the trestle. Approximately it can be considered that the single track piers have a width of 3 m, and a doubletrack, 6 m.

Just as for bridges, in trestles it is possible to distinguish span structure and support ones. The principles of bridge building are also applicable to trestles. However, trestles usually do not cross water obstacles and they differ considerably in comparison with bridges by the variety of abutments and loads and by a more complex form in the design. Thus, trestles have not only right angled, but also the C-shaped and other outlines by design. As for the equipment, the trestles can support, just as the deckhouse buildings, electric locomotives and trolleys, special carts, the hoppers, funnels and troughs, the tippers of various types and the compensators of the height.

In connection with the comparatively complex form of trestles by design it is necessary to focus special attention on the connections, which assure the spatial immutability of the supporting structures.

The joints of the abutments of the trestle and other construction and dams are of significant value. Here, one ought to assure the smooth passages of rolling stock from the more rigid construction to the less rigid, and vice versa.

During building and operation of the trestles it is necessary to provide a number of measures for safety: one should provide for the passages of people, passage to the outside of over-all components, and all trestles should be equipped with guards, exits, stairs and so on.

The trestles, which possess a shelter, usually are called galleries. Haulage galleries differ from trestles not only by having shelters, but also by the conditions of their operation. If, for example, haulage trestles are arranged with a ride toward the top, then they have the supporting structures, sheltered in over-all sizes of the construction and, consequently, protected from atmospheric pressure.

The span structure of the trestle usually has two main beams. With single track trestles these beams support a load from the ties, the plates, resting directly on them. With wide trestles transverse and longitudinal beams or intermediate main beams are introduced. With a limited value of the interval of the transverse frames of the trestles (6 m, and sometimes 12 m) the direct placement of the ribbed slabs to the crosspieces of the transverse frames is possible. In these cases main, transverse and longitudinal beams can be excluded.

The diagrams of the span structure substantially depend on the designs employed. With metallic span structures transverse and longitudinal beams are quite frequently used. With reinforced concrete sectional span structures the simpler diagrams are select resulting in the least number of type and size of sectional designs.

Fig. 224 shows the well distributed schemes of span structures of the trestles.

In Fig. 224a - a diagram with two main beams 1, connected to 2 and ties, lying directly on the main beams. In Fig. 224b - a diagram, which corresponds to the wider trestle and to the utilization of complex designs. Above the transverse beams are the wooden longitudinal beams with small spans and a small section. Ties are located on the longitudinal beams.

brace is erected in the axis of the transverse beam with a bracing of the beam of the second span. The second brace of the second span is projected beyond the face of the transverse beam. The entire pack is united on the supports and is fastened with bolts to the transverse beams. Figure 226 shows a variant of the uprights 3 of the supporting steel frames in the form of steel pipes. The given variant is composed based on local conditions and exists in full size with relatively complex outlines of a trestle by design. In this case the elements 4 are introduced only in separate complex supports.

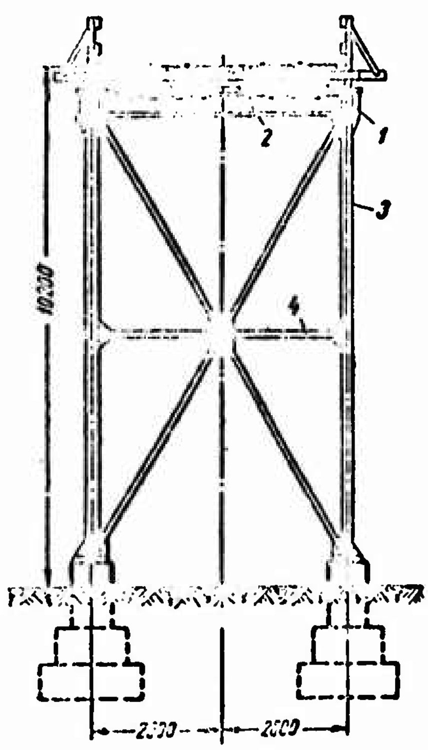

Fig. 226. The trestle of a combined design: 1 - main beam; 2 - transverse beams; 3 - upright; 4 - connections.

Under conditions of simpler outlines of a trestle by design and its straightness one could make the pier of a combined design with the preferred utilization of precast reinforced concrete. The supports of such a trestle can be formed by ferroconcrete frames or uprights with metallic connections. The main and transverse beams of a span structure can be joined into one general part.

The weight indexes of such a trestle are more favorable with spans of 8-12 m and more, and at its width is increased somewhat.

Figure 227 shows a sectional ferroconcrete trestle. The transverse frame of the trestle is composed of ferroconcrete uprights, which do not have uprights along its length, with crosspieces, lie on the cantilever of uprights, and with metallic lightweight connections. The upper crosspiece is simultaneously the element, which increases the stability of the main beams. In a longitudinal direction the trestle has a connection in each expansion block. The whole trestle thus consists of three type and dimension sectional ferroconcrete parts.

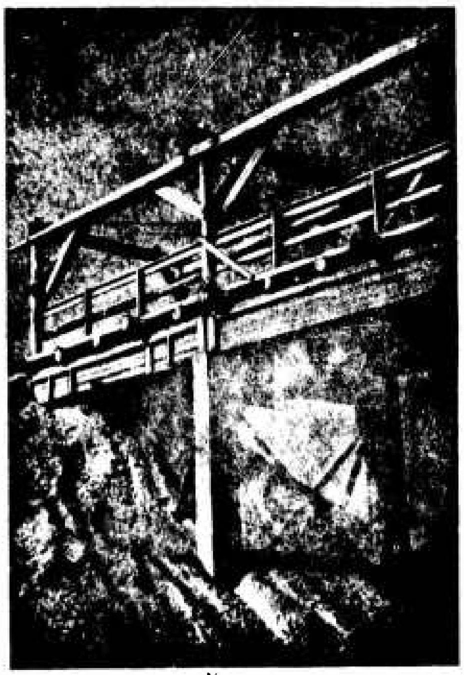

Fig. 227. A view of a sectional ferroconcrete trestle.

With a height of a trestle of about 9 m and trolleys with a load-carrying capacity of 2.5 t, the expenditure of precast reinforced concrete constitutes 0.5 m^3 per 1 m^2 of area of the trestle.

2. Haulage Galleries

The purpose of haulage galleries is usually the same as that for haulage trestles. Just as with trestles, haulage galleries serve to haul the rolling stock from a deckhouse building to the loading funnels and hoppers as well as for other purposes.

Haulage galleries frequently are a continuation of the deckhouse building, and in most cases, they differ from the deckhouse building itself only by the considerably less over-all sizes of the transverse section.

In a number of cases the sections of a deckhouse building and haulage galleries can hardly be divided. This is observed when the stem of the mine shaft adjoins directly the haulage galleries (see Fig. 205). As a result of that shown, in connection with the presence of guards in the planes of the walls and flooring, the degree of the stability of the haulage galleries frequently corresponds to the treatment of the deckhouse buildings.

The haulage galleries are usually of more durable construction in comparison with trestles and they are used in larger mines. The sizes of the galleries exceed the sizes of the trestles. The height of the overwhelming majority of galleries is within the limits of 15 m, i.e., within the limits of the height of the haulage trestles, but there are haulage galleries having a height even to 18-24 m above the level of the surface. Thus, for instance, the deckhouse building and the haulage galleries from one of the ore mines (Fig. 228) are characterized by haulage at heights of 20 m above the planning level.

The haulage of cold galleries usually of smaller height frequently are made with complete or partial utilization of wooden support designs.

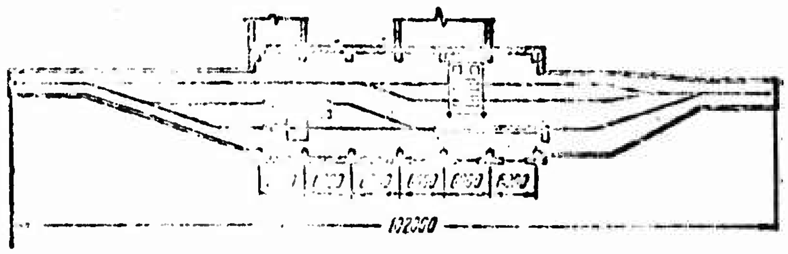

Fig. 228. Galleries of an ore mine with haulage at heights of 20 m above the level of planning.

The galleries, adjacent to the deckhouse buildings or directly at the stem of the mine shaft and having capital external guards, are usually equipped with heating from a central boiler or from an adjacent calorific device. In this instance the measures, given in Chapter VIII are completely applicable to the problem related to the ..aulage galleries.

The most widely used width of haulage galleries: 3, 6, 9, 12 m and up to 30 m. The spans of the main trusses and beams are shown in Table 32.

Table 32.

Material of flights	Span of galleries with span structures made of trusses, m	Span of beam galleries, m
Wood	6-20	3-6
Reinforced concrete	18-30	6-18
Steel	up to 50 and above	–

The last sizes of the spans correspond to cases of the arrangement of the galleries in ore stock piles, in the receiving funnels of ore treatment combines, and also to cases of crossing buildings

and structures under compact conditions. Flights 18, 24 and 30 m are convenient in adjoining haulage galleries at the mouths of the stems of mine shafts, with soils with low bearing capacity and in a number of other cases.

With the ferroconcrete beam support structures of the trestles usually the ride is set at the top. With metallic support designs (trusses) with spans from 18 m and above, the ride is set downwards.

The placing of the main beams and trusses, transverse beams, longitudinal beams, support loads from rolling stock, is figured in reference to the schemes, described in 1 of this chapter, partially given in Fig. 224, and also in reference to the data in Chapter VIII.

With narrow galleries having a width, for example, of 3 m, the transverse beams are not used. With wider galleries transverse beams of 3-6 m are used which is expedient with steel support designs of galleries and inexpedient with sectional ferroconcrete designs. With monolithic designs with the utilization, for example, of support reinforced frames and sectional ferroconcrete designs along with necessary cranes, the utilization of transverse beams, joined to the main beams of the span structure (Fig. 229a) is expedient.

The foundation of the haulage galleries - predominantly ferroconcrete, monolithic and sectional. The uprights of the supporting frames of the haulage galleries in most cases can be accepted as sectional ferroconcrete according to the diagram shown in Fig. 227. The joints of uprights can be metallic, which substantially simplifies the solution of the gallery.

The main beams and slabs of the span structures with the spans of the main beams at 6, 9, 12 m, sometimes 18 m can be made in precast reinforced concrete. The spans of 12-30 m can be expediently made with stress-shockproof structures.

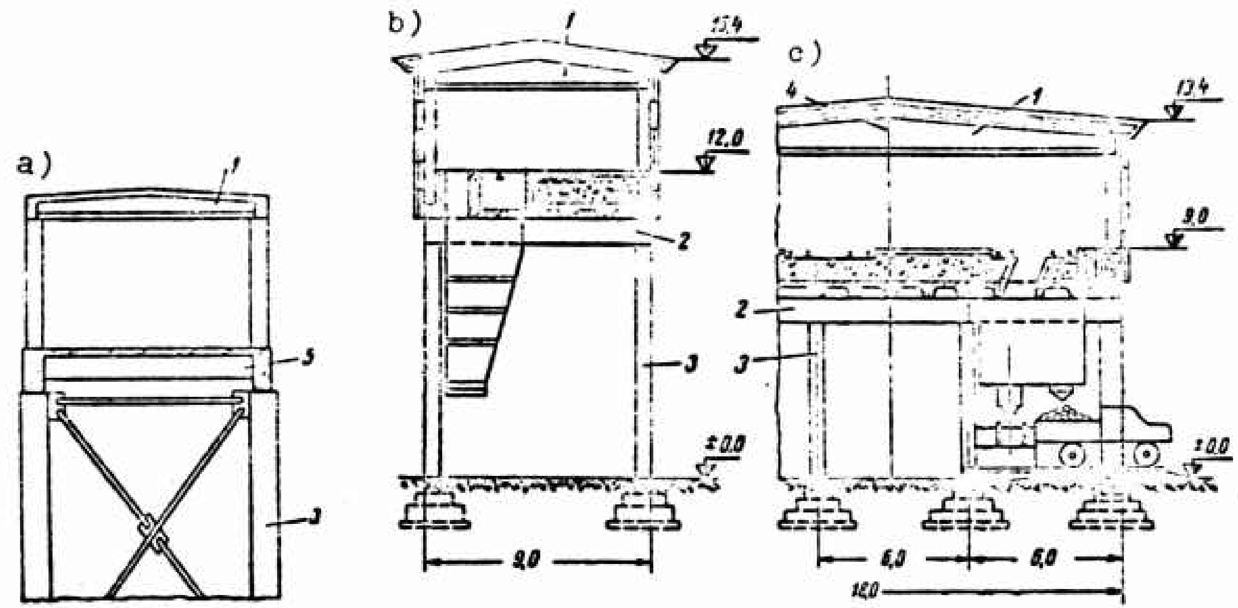

Fig. 229. Sectional ferroconcrete haulage galleries: a) haulage gallery with units of the main beam and transverse flooring beams of a span structure and associated supports; b, c) haulage galleries with non-beam flooring and frame supports in the section of the unloading; 1 - capping beam; 2 - crosspiece of the main frames; 3 - bars of the main frames; 4 - axis of symmetry; 5 - unit of the main and transverse beams and the slabs of the flooring of the span structure.

The frame of the galleries itself using reinforced concrete, can be made from lightweight sectional uprights, set 6 m apart and transverse crosspieces (at a level of capping), in the capacity of which should serve as standardized sectional ferroconcrete and prestressed beams with nominal spans of 6-18 m and more.

If necessary for ballast, with the limited height of the construction and with the possibility of introducing a six-meter spacing of the uprights, in a number of cases, the utilization of flooring slabs, directly on the crosspieces of the frames (Fig. 229b, c) is expedient.

It should be kept in mind that this design is expedient with the spacing of the frames equal to 6 m, and is not to be excluded with large spans of slabs.

For ferroconcrete flooring of haulage galleries in the cases noted here special ferroconcrete and prestressed capping plates with spans of 6 m under a load of 5-12 t/m^2 and more are used. In sections with a reduced load using lightweight trolleys and in the presence of ballast the utilization of standarized slabs with a nominal length of 6 m for the flooring of industrial buildings under a load 2 t/m^2 and more, is possible. With various complex designs there is use for ferroconcrete flooring slabs with spans of 3 m. The data on the utilization of sectional ferroconcrete designs in the capping of deckhouse buildings wholly pertain to haulage galleries. Here, cappings of large ferroconcrete slabs, 6 × 3 (1.5) m, in sizes with spans of 12 m are widely used for sectional ferroconcrete and prestressed beams.

Although the haulage galleries are quite extensive, because of the small width of the galleries, the problems of natural illumination solve themselves without any specific difficulties. This makes it possible to place the window openings in the most convenient way from the point of view of using large wall panels. In this case it

is entirely possible to assign the width of the openings as multiples of 6 m, i.e., to use continuous glass for the length of the panels, from two or from one side of the gallery. One-sided continuous glass and separately arranged louvers or sashes in the opposite wall of the gallery are used to make possible through ventilation. These measures guarantee the most successful solution of the wall filling of galleries.

With the resolution of the problems enumerated here the wall filling can stand using the minimum of type and dimension of cold or warm large wall panels.

The data on the utilization of sectional wall panels in mine construction are also quite valid for haulage galleries. In the galleries the possibility of the failure of the light-weight wall fillings with the derailment of trolleys should be taken into account. In connection with this, in the galleries, just as in deckhouse buildings, it is expedient to use warm wall panels with support and covering slabs with the arrangement of the dense and rigid ferroconcrete and prestressed support slabs from the inner side of the wall barriers.

In cold galleries the utilization of heavy ferroconcrete and prestressed wall slabe at nominal sizes 6.0×1.2 m, and also slabs 12 m in length, is possible.

Just as in deckhouse buildings, the walls of the haulage galleries are made using cold sheathing of asbestos-cement corrugated sheets of a reinforced profile, and also warm shields, faced on two sides with asbestos-cement corrugated wavy sheets with effective heat insulation placed on the inside. One ought to take into account that the velocity of the motion of rolling stock in haulage galleries is usually higher in comparison with deckhouse buildings; therefore, the protection of the walls with stop rails or corners is provided over the entire length of the galleries.

Just as in the surface buildings and trestles, the floors of the haulage galleries are usually characterized by inclines and drops, by the presence of tracks and considerable loads from rolling stock, suspension funnels, troughs, unloading equipment and so on.

Compensation for the inclines of the passages are made because of the layer of ballast. The height of the ferroconcrete designs in this case within the limits of one unit remains constant.

The heads of the rails are usually somewhat raised over the level of the floor. The adjoining construction of the tracks to the floors is done on Fig. 218.

The separate examples of haulage galleries are noted below.

Figure 230 shows a haulage gallery using wooden support designs. The trestle is composed of transverse frames, spaced 4 m apart with the same span of the main beams of the construction. The light-weight frames of the hip of a gallery from the planks with the nails are installed every 2 m. The lathing under the roofing made from asbestos-cement corrugated sheets rests directly on the crosspieces of the frames. The foundations of the construction - concrete posts.

Figure 231 shows a haulage gallery using sectional ferroconcrete elements in conjunction with light wooden structures. The upper structure of the gallery has two sections of a different width. In one case the width constitutes 6 m, in another, 4 m. In both cases the transverse ferroconcrete frames of the construction are identical - a width of 4 m. The distinction in the width of the hip of the gallery is attained because of the wooden designs, the various frames of the hip and the presence of small cantilevers, supporting these frames at a width of 6 m.

The uprights of a sectional ferroconcrete transverse frame are furnished with sockets of the bootleg type. The crosspiece of

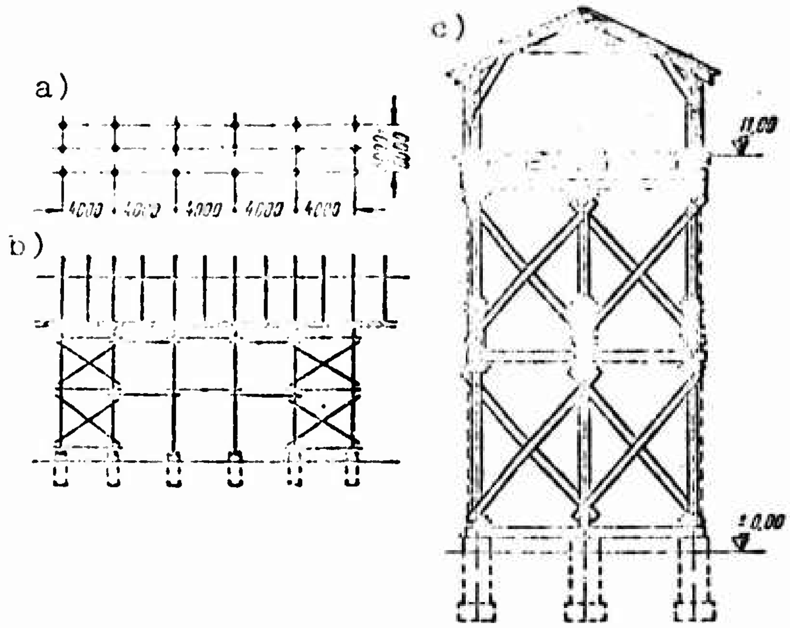

Fig. 230. A haulage gallery with wooden support structures: a) the plan of the uprights; b) longitudinal section cut; c) transverse section.

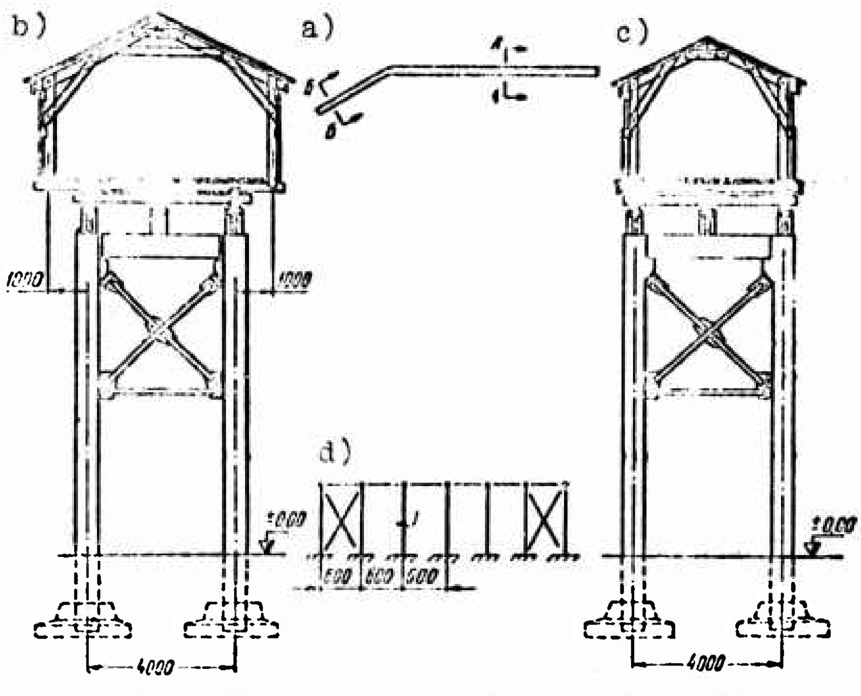

Fig. 231. The haulage gallery of combined design different width: a) planning design; b) gallery at the widened section (section A-A); c) gallery at a width of 4 m (section Б-Б); d) profile design of a section of the lower structure.

a frame supports the load of one of the main beams. The two remaining beams rest directly on the uprights of the frame. The transverse rigidity of a frame is secured (besides the sealing of the uprights in the shoes) by the presence of lightweight steel joints from the corners in the upper part, working under tension. The presence of the joints results in a substantial reduction of the bending moments in the joints and rods of the frame, specifically in the uprights of the frame, and also results in a very substantial simplification of the joints. In this case it is not necessary to produce high rigidity of the joints of the frames, attainable by the introduction of comparatively complex and developed matching parts. The rigidity and stability of the trestle in a longitudinal direction are provided by introducing the joints of uprights (Fig. 231d) within the limits of the expansion block. As a result of using the described design, an extremely small number of type and dimension sectional ferroconcrete elements were needed. Despite the presence of two variable sections, which are distinguished by the width of the gallery, for the realization of the described haulage galleries (Fig. 232) only three sectional ferroconcrete elements are necessary: the upright, crosspiece of a transverse frame, and the main beam. For a similar construction such an amount of type and dimensions of sectional elements is a record.

Fig. 232. View of the gallery during the period of the completion of the building.

At a height of a gallery of about 9 m and trolleys with a load-carrying capacity of 2.5 t, the expenditure of precast reinforced concrete constitutes 0.4 m^3, steel joints and beams, 0.03 t, wood 0.24 m^3 per 1 m^2 area of floor of the gallery.

One ought to show that in the absence of the need for a variable width of the hips of the galleries, it would be possible to make without any specific difficulties the sectional ferroconcrete hip of the gallery. The diagrams of the simplest all-reinforced concrete haulage galleries are given in Fig. 229.

The given solutions of haulage trestles and galleries provide a basis for showing the possibility of their unification using sectional ferroconcrete designs. In the first place unification should include the haulage trestles and galleries with a constant spacing of the transverse frames, equal to 6 m.

With the large spans of the main beams or trusses as well as with the complex forms of galleries, and also with the reconstruction of the gallery it is expedient to build using steel designs. Precast reinforced concrete in such cases is represented by various capping slabs, flooring and wall panels.

Figure 233 shows a longitudinal section, and in Fig. 234, a transverse section of a part of the haulage gallery of a large iron-ore mine, made under conditions of the need to utilize the spans of the main trusses of construction within the limits up to 21 m and predominantly equal to 18 and 21 m with the ride downward. The adjoining section of the gallery, where steel support structures are also used, have spans, predominantly equal to 12 m. Here, the trusses for the ride are set upward. All the span structures have transverse beams, spaced at a distance of 3.0-3.5 m, riveted on supports in the uprights of the transverse frames (with the formation in this way of cross pieces of the latter), and in spans - in the uprights of the trusses.

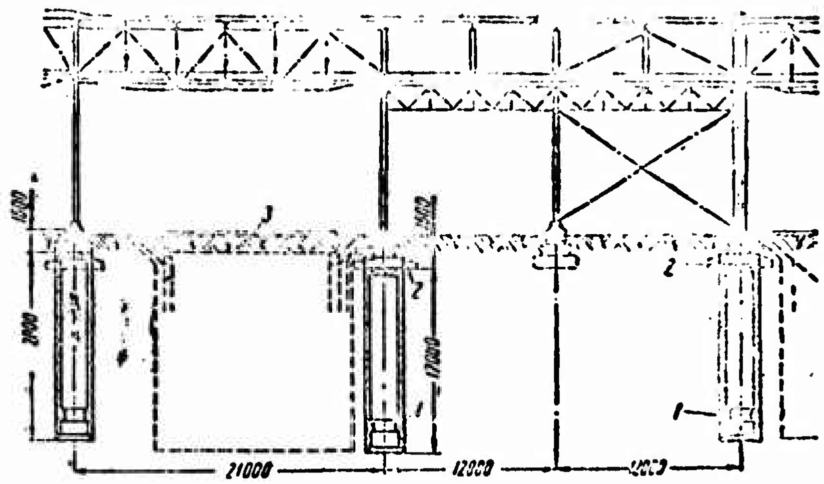

Fig. 233. Longitudinal section of a metallic haulage gallery: 1 - moveable caisson (variant of the equipment of foundations; 2 - developed base plate (existing variant); 3 - existing receiving funnel.

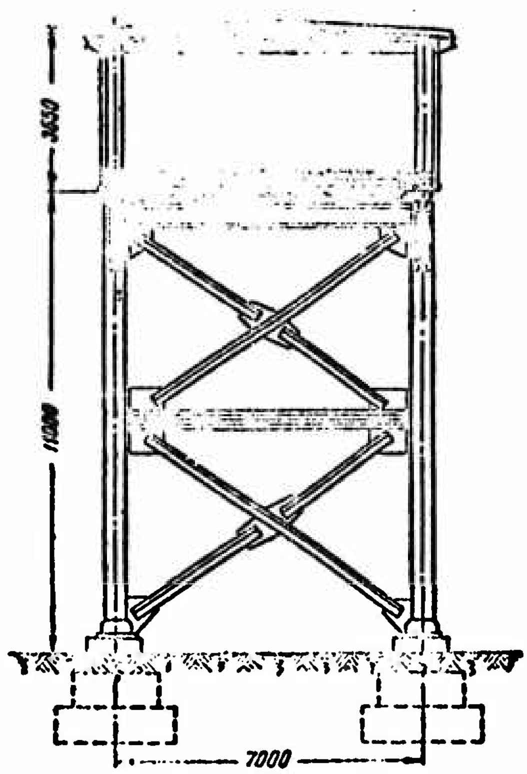

Fig. 234. The transverse section of a haulage gallery.

All the transverse beams are rolled double-T No. 55a beams and are located at one level. Because of this, and also in view of the direct bracing of 12-meter trusses of span structures to the uprights of frames, the main trusses with a span of 21 m, which are centrally installed relative to the supporting uprights, have polygonal outlines (Fig. 233).

The sectional ferroconcrete ribbed slabs resting on the longitudinal flooring beams of the gallery, located above the slag, are considered as heat insulation, ballast and clean floor.

The filling of the walls in connection with the considerable extent of the galleries is regarded as the shields, coldproof slag mats and are faced on two sides by asbestos-cement sheets having a reinforced profile. This filling, to a sufficient degree, is effective from the point of view of weight. One ought only to keep in mind, that under the conditions of haulage galleries the continuous protection of walls with metallic rods at the level of the body of the trolleys is necessary. A more capital solution of the walls of haulage galleries, just as in a number of other constructions under mine conditions, are the ferroconcrete heavy wall panels.

The haulage galleries, built under comparatively complex conditions, with considerable spans and loads, associated with the haulage of 10-ton electric locomotives and components of trolleys with a load-carrying capacity of 2.5 t, are characterized by average indexes of expenditure of metallic-structures of 1.6 t/m and about 0.3 t per 1 m^3 area of construction. The given indexes, considering the complexity of a gallery, are satisfactory. Over a considerable extent of galleries the standard gauge railway is located under them, which results in the utilization of frame supports. Several sections of the gallery have slanting abutments. In the composition of these galleries there are also frames, installed along the circumferences. The central subassembly of the construction of the stem of the mine shaft, in this case, was also comparatively complex. Finally, in the adjoining galleries

leading to receiving funnels of tne crushing mill, built several years before the building of the galleries, the supports of construction are located on fill. Figure 233 shows the variants of equipment of the foundations of galleries in these sections. The foundations 2 are presented to scale as developed by design of ferroconcrete ribbed slabs, placed on fill.

Such conditions complicate the equipment of haulage galleries and result in producing considerable difficulties pending on the sectional ferroconcrete designs. Under such complex conditions (unlike the above mentioned solutions of all-reinforced haulage galleries) one ought to recognize how expedient some combined designs are, when the sectional repeating parts are made from reinforced concrete.

Figure 235 shows a part of a longitudinal section, adjacent to a deckhouse building - the haulage galleries of the capital mine shaft. The galleries are calculated for the car haul with a load-carrying capacity of 10 t with the overall weight of the trolley with the ore is 15.2 t. With an overall length of 151.5 m, the spans of the galleries constitute 12 m and sometimes 24 m. The distance between the main trusses are 6 and 9 m. The supports of the trusses are sectional ferroconcrete columns, unitized in the containers of the ferroconcrete shoes. The stability of the columns is provided by metallic joints, installed within the limits of each expansion block. Besides the ferroconcrete columns the following sectional ferroconcrete and reinforced gas-fired concrete designs have been used in construction:

sectional ferroconcrete capping slabs with nominal sizes, 1.5 × 6.0 m;

sectional ferroconcrete flooring slabs with nominal sizes, 1.5 × 3.0 m;

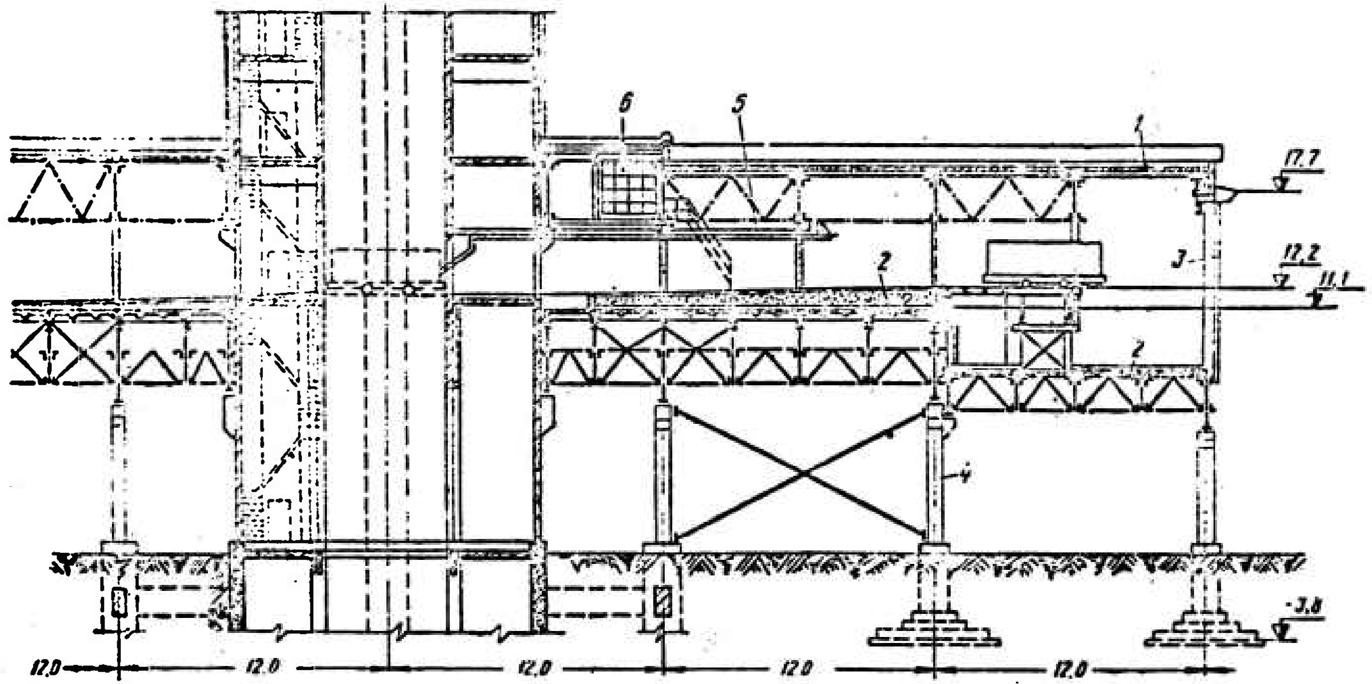

Fig. 235. The haulage gallery of a capital ore mine shaft on junction plates with a deckhouse building, a longitudinal section: 1 - sectional large-size capping slabs; 2 - sectional flooring slabs; 3 - sectional wall panels; 4 - sectional ferroconcrete uprights; 5 - pushers; 6 - control station.

sectional ferroconcrete broadside flooring beams, which are divided into sections of haulage galleries with various levels of flooring (Fig. 236);

sectional reinforced gas-fired concrete wall panels with nominal sizes, 1.2 × 6.0 m.

The expenditure of steel designs of haulage galleries amounts to about 0.3 t, the expenditure of sectional ferroconcrete designs - 0.3 m^3 to 1 m^2 of construction.

With the possibility of establishing a uniform distance between the trusses, equal, for example, to 6 m, it is expedient to increase the degree of utilization of the sectional ferroconcrete structures of the construction. Thus, for instance, in a haulage gallery (see Fig. 235) in this instance made from metallic structures it is possible, specifically, to maintain only the steel cappings and flooring trusses. All the remaining designs, including sectional flooring slabs with nominal sizes, 6 × 1.5 m, the capping slabs sectional broadside beams, sectional columns can be made of ferroconcretecrete, but the sectional wall panels - using sectional ferroconcrete or prestressed slabs, and also using reinforced cellular concrete. The expenditure of metal, necessary for the erection of construction according to the described variant, is shortened, the cost of the construction is reduced.

A further step in the way of reducing expenditure of metal when building haulage structure is the utilization of all-reinforced concrete structures of the haulage galleries. With complex forms of galleries by design, and numerous adjoining structures and other local complications of construction, it is expedient to apply combined designs with a preferred introduction of precast reinforced concrete and prestressed designs and the local inclusions of steel

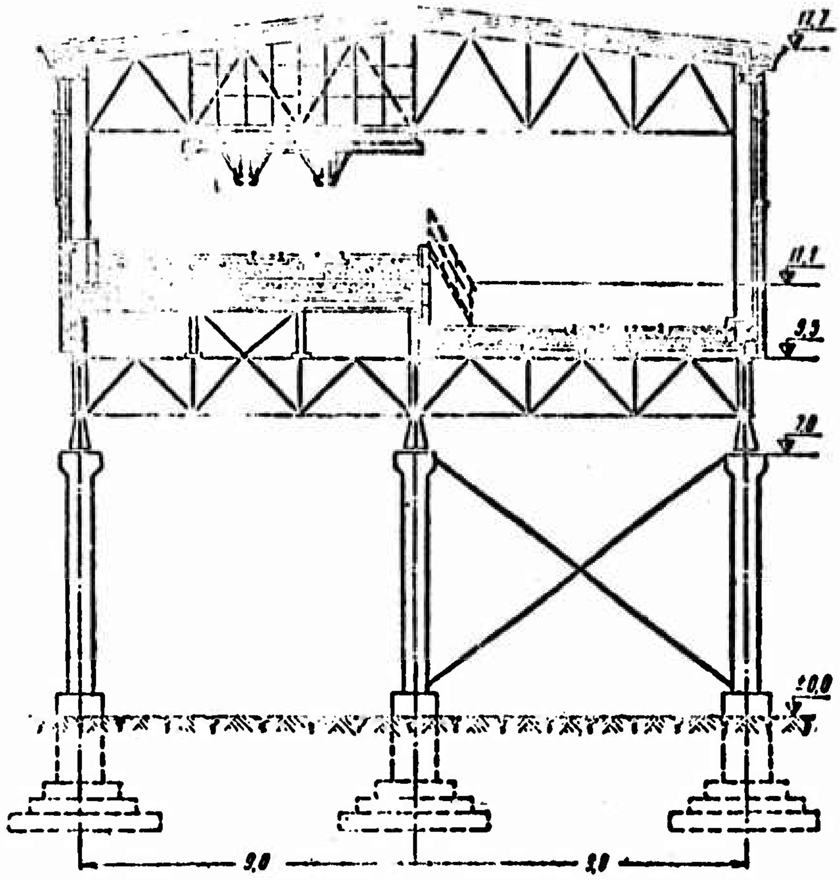

Fig. 236. The transverse section of a section of a haulage gallery of a capital ore mine shaft.

designs in the roundings, in the angles of construction, in separate nonstandard spans, superstructures and so on. In particular cases with simple forms of construction sectional all-reinforced haulage galleries can be characterized by Figs. 216 and 217 (under the condition of the exception of the wall filling lower than of the level of the flooring slabs). In this instance with moderate loads and with trolleys having a load-carrying capacity up to 4-5 t, the expenditure of precast reinforced concrete constitutes about 0.6 m^3, and the expenditure of reinforced cellular concrete, about 0.2 m^3 per 1 m^2 area of construction. One ought to note that the latter indexes substantially change with a change in the width of the construction.

3. The Calculation of the Construction of Haulage

During the calculation of the designs of haulage trestles and galleries, specific attention should be given to loads, acting on them.

Among the special loads of construction of haulage, there are mainly the loads from rolling stock. Just as in deckhouse buildings, 7- and also 10-ton electric locomotives here are used, and also electric locomotives of a large weight with trains of trolleys having a load-carrying capacity of 2.5-10 t (capacity of 1-4 m^3) and others. The trestles and galleries are loaded with trains along with loaded or empty trolleys. The effect of vertical loads from rolling stock, the calculation of coefficients of dynamicity, the effect of the braking of the rolling stock, other loads, the values of the coefficients of overload are taken just as for deckhouse buildings.

The coefficients of overload of mobile load of span structures of trestles and galleries of capital mine shaft are taken equal to 1.3-1.4. In this case the lesser values correspond to piers, and the larger - to haulage galleries. The coefficients of overload for a mobile load of ferroconcrete span structures of haulage trestles

and galleries are equal to 1.4 and 1.5. With a load of two, three and more tracks, the values of the coefficients overload are reduced by 10-20%.

A dynamic coefficient $(1+\mu)$ with haulage of the car having an overall weight of the loaded vessel of less than 20 t is taken according to instructions in 5 of Chapter VIII.

The dynamic action pressure of the empty stocks and temporary horizontal loads are not considered. The horizontal loads during the braking of the rolling stock are taken equal to 10% from the value of the vertical loads of rolling stock. The number of braking rolling stock corresponds to the number of turning electric locomotives.

The landings outside the over-all sizes of mobile rolling stock are usually loaded having a temporary evenly distributed load with the coefficient of overload of 1.2 and 1.3.

Constant loads are predominantly considered with the introduction of the coefficient of overload of 1.1, and the determination of the weight of the ballast - the heat of isolation - with the coefficient of overload of 1.2. Assembly loads in the galleries and trestles usually are not considered.

The effect of blasting is considered accordingly to data presented in Chapter II.

C H A P T E R X

CONVEYOR GALLERIES

1. General Information

Conveyor galleries are widely used in enterprises of various lines: mining, coal, coke, cement, fireproof materials industry, nonmetallic, etc.

The chief data are noted below on conveyor galleries, the attempt to bring to light their building and operation in the iron-ore industry, and recommendations in the way of diagrams and designs of the galleries are presented.

Conveyor galleries in mines serve as the space for one or two, and more rarely three conveyors. Their over-all sizes by design are determined by a number and by the sizes of the installed conveyors and by the width of the passages between their frames and walls of the gallery.

The width of a conveyor, determined by the width of its frame, depend on the width of the belt, which usually constitutes 500 up to 1200 mm. With sufficient accuracy it can be said that:

at belt	width	of	400 mm	frame	width	is	0.80 m
"	"	"	"	500 mm	"	"	"	0.95 m
"	"	"	"	650 mm	"	"	"	1.05 m
"	"	"	"	800 mm	"	"	"	1.25 m

at belt	width	of	1000 mm	frame	width	is	1.50 m
"	"	"	"	1200 mm	"	"	"	1.75 m
"	"	"	"	1400 mm	"	"	"	1.95 m
"	"	"	"	1600 mm	"	"	"	2.15 m

The frame of conveyors with belts 500-800 mm is wider than the belt by 0.4-0.45 m, with a belt 1000 mm - by 0.5 m, and with the widest belt - to 0.55 m.

The passage from one side of the conveyor, including the overall size of the heating devices, usually constitutes 1-1.2 m, the repair clearance between the conveyor and the plane of the wall - 0.5-0.7 m.

There is a proposal about the width of a one-sided passage being 1.0 m, the repair clearance being 0.5 m and each of the two-sided passages being 0.75 m. The width of the conveyor frames is proposed to be multiples of 0.5.

In this instance:

for belt	width	of	400-500 mm	frame	width	is	1.0 m
"	"	"	650-1000 mm	"	"	"	1.5 m
"	"	"	1200-1400 mm	"	"	"	2.0 m
"	"	"	1600-2000 mm	"	"	"	2.5 m

The general width of the gallery is also assumed as having a modulus of 0.5 m and with one and two conveyors, it constitutes:

for belt width	400-500 mm	- 2.5 and 4 m
" " "	650-1000 mm	- 3.0 and 5 m
" " "	1200-1400 mm	- 3.5 and 6 m
" " "	1600-2000 mm	- 4.0 and 7 m

A part of the last sizes is substantially more than the usual sizes of the width of galleries for commonly used conveyors. Thus, for instance, two conveyors with belts having a width of 650 mm can be placed in a gallery having a width of 4.1 m. On the other hand, the width of a gallery of 4 m for a conveyor with a belt having a width of 2000 m, is inadequate.

On the basis of this, it can be shown, that for single conveyors galleries of the following width are necessary:

3.0 m	-	for commonly used conveyors with belts having width of 650, 800 and 1000 m
3.25-3.5 m	-	for conveyors with a width of the belt of 1200 and 1400 m
4.0 m	-	" " " " " " " " " 1400 and 1600 m
4.5 m	-	" " " " " " " " " 2000 m

For two conveyors galleries of the following width are necessary:

4.0 with belt width	500 mm
4.5 " " "	650 and 650; 650 and 1000; 800 and 800
5.0 " " "	650 and 1400; 800 and 1200; 1000 and 1000
5.5 " " "	1000 and 1400; 1200 and 1200
6.0 m with working passage from 1.1 m and wider	1400 and 1000; 1400 and 1200; 1400 and 1400; 1200 and 1200

It is recommended to assign 2.5 m as the height of the galleries and this value is apparently oversize, since in practice, the height constitutes 2.0-2.3 m. A height of 2.0 m is used in the absence of pipe lines, a height of 2.2-2.3 m is sufficient even with the installation of lines over the passage and repair clearance. Independent of this, a height above the belts to the ceiling of the gallery should constitute not less than 700 mm under usual conditions.

The given data about the height of galleries pertain to the bulk of cases for equipment in inclined and horizontal galleries with the transport of material by conveyors, which do not possess the automatic or other dumping trolleys. With dumping trolleys the height substantially increases and it is set accordingly to the specific tasks.

The covering of the belts is done only in specific cases with the transport of dust raising material, with vapor-releasing materials and in some other cases.

Equipment made of rails or handrails, strengthening the frames

of conveyors, higher than the level of the belts on each side is recommended. The presence of handrails is equal to a substantial widening of the passages and to an increase in the degree of industrial safety. For convenience and safety the movements of the operating personnel, the walls inside the gallery are also equipped with hand-rails, but on the roofing metallic rails are installed.

For the frames of the conveyors on the floor of the gallery bars are laid; in necessary cases they are provided with matching parts.

As a rule, in galleries one should provide lining for the water pipes with faucet fixtures and provisions for flushing the floors, walls and ceilings with organized water outlets.

Smooth pipes, set along the outer walls are used to serve heating devices. It is not recommended to fasten the pipes to the frames of conveyors, since this would require their frequent cleaning of an ore and dust and it hampers the servicing of the heating system.

Electric cables are suspended on insulators to the roofing above the repair clearance.

2. Schemes and Designs of Galleries

The safety guards of the galleries should have a minimum weight; therefore the filling of walls with masonry of brick and slag blocks should not be used. For filling the walls it is expedient to apply sheathing with light heat insulation contained between the two asbestos-cement sheets. During unfavorable temperature-moisture conditions, the walls of galleries are in operation using warm panels composed of supporting ferroconcrete prestressed slabs and light-weight slabs providing the simplest protection against moisture.

The flooring of the galleries in all cases are made of ferro-concrete and prestressed sectional slabs. The designs of the capping in the most projects are analogous to the designs of the flooring.

In a number of cases light-weight designs are also used, based upon the introduction asbestos-cement sheets, asbestos-cement shields and panels.

The data below mentioned about the weight of conveyor galleries characterize the value of supporting designs and the designs of guards. The weight indexes of the structure, and also the indexes of the expenditure of the steel and the degree of industrial designs wholly depend on the selection of these elements.

The spans depending on the height of the construction and employed designs change within relatively large limits. The spans of wooden galleries usually constitute 5-6 m, and sometimes with span structures made of trusses extended to 15-20 and 30 m. The spans of galleries using ferroconcrete and combined designs are equal to 6-18 m, but sometimes reach 85 m. The spans of metallic galleries are frequently found within the limits of 18-24 m; they also are used in spans up to 30 m. The spans of steel galleries frequently reach 50-60 m, sometimes 100 m. With an increase in the height and length of a gallery, in the degree of constriction of the landing and in the complexity of the foundations and bases, the significance of the spans increase.

It is necessary to note the large variety of designs, particularly the lengths, spans, inclines and abutments of galleries to adjacent construction, which in a number of cases, strongly complicates and hampers the utilization of sectional ferroconcrete prestressed designs. Under these same conditions the replacement of metallic supporting designs with heavier ferroconcrete ones frequently does not provide a substantial reduction of the expenditure of the metal. Therefore, even today rather broad utilization of the steel designs of galleries is observed.

Metallic conveyor galleries. The optimum spans of the steel trusses of inclined galleries with light-weight guards (with ferroconcrete slabs of flooring up to 3 m in length) and with an overall

length of the gallery of 60 m are equal to 15-20 m, and when faced with difficulties particularly with foundations and the interception of communications - 18-24 m. Under the same conditions, but at less angles of slope of the galleries, amounting to 0-16° to the horizon, the optimum span of the trusses depend on the average height of the columns and it exceeds this amount by 15-20%. With the height of the floor of the gallery over the ground surface of 15 m and higher, the optimum spans of trusses are equal to 18-24 m. Using these spans the least sum total expenditure of steel is assured which is necessary for the manufacture of the supports and span structures. The minimum expenditure of rolled steel for an inclined gallery with light guards having a width of 3 m (with the flooring slabs up to 3 m in length) with optimum spans and a length of 60 m constitutes 0.15 t/m^2, but at a length of 90 m - 0.17 t/m^2 of the area of the pier. The expenditure of steel increases approximately up to 10% with the partial utilization of heavier guard designs, for example, with the introduction of ferroconcrete slabs in the flooring and capping. The indexes of the expenditure of steel diminish approximately 20% with galleries 6 m in width. With wide galleries and with an arrangement of the conveyors at the top of the trusses (Fig. 237a) a certain decrease in the expenditure of steel designs is possible because of the lighter weight of the transverse beams of the galleries with their smaller span and partly because of the reduction of the height of the supports. When dealing with galleries of a comparatively small width, equal to the distance between the trusses, then the weight of steel designs according to variants b and c (Fig. 237) will practically be equal. In this case the weight for variant c (Fig. 237) can be less in comparison with variant b. This can be explained by the preferential utilization of galleries with conveyors on level with the lower flanges. The arrangement of conveyors towards the top of the trusses is recommended with high humidity in the gallery.

One ought to show the considerable details of the equipment of conveyor galleries with metallic supporting structures.

The frame of the galleries should be invariable in longitudinal and transverse directions.

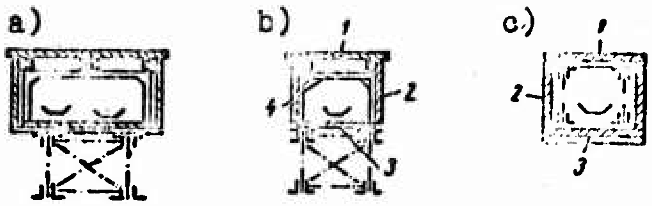

Fig. 237. Diagrams of conveyor galleries: a) wide gallery with the arrangement of conveyors towards the top of the trusses; b) narrow gallery with the arrangement of the conveyor towards the top of the trusses; c) narrow gallery with the arrangement of the conveyor towards the bottom of the trusses; 1, 2 and 3 - the guards for the capping, walls, flooring; 4 - frames of the hip of the gallery.

Lengthwise to the construction the fixed and movable supports in accordance with the placing of the expansion joints are provided. Distances between the expansion joints are taken within the following limits:

for the heated galleries with loads from the conveyors along the lower strap, the length of the temperature block should not exceed 150 m; the presence of the open or half-open straps frequently results in the reduction of the length of the expansion block;

for unheated galleries with the cold guards of the walls and flooring, the distance between the expansion joints should not exceed 120 m, but for practically open galleries with large breaks in the guards and with the partial or complete absence of guards the distance is taken within the limits of 90-100 m.

The ratio of the span and the height of the trusses of the galleries constitutes 10-12. With a lower arrangement of the conveyors the height of the trusses frequently is taken equal to 2.4-2.8 m, which is determined from the condition of sufficient height of the passages.

The fixed supports of the trusses of a span structure are predominantly set at the lower ends of the expansion blocks of the inclined span structures and they are usually installed on concrete and ferroconcrete foundations and uprights as well as on steel tower supports.

The upper supports of the trusses of galleries are mobile, with the utilization of tangential and partly other supports with the linear transmission of the loads and flat supports, but with spans of more than 25 m and with the support of the span structures on stone walls - rolling supports. With a support on a steel or ferroconcrete frame and allowing for of the forces of friction the utilization of tangential and flat movable supports is possible. Intermediate steel planes in the direction of the length of the structure of the support of the galleries are considered as pendulum uprights, hinge-connected with the trusses and foundations. The mutual fastening of the trusses and intermediate supports are usually made on bolts without the possibility of displacements of the trusses relative to the uprights. In the presence of intermediate pendulum uprights, the rolling support is installed at the end of the expansion block.

The trusses of the galleries are used with parallel straps and ascending braces of the supporting panels. The grating of the trusses are triangular and diagonal shaped.

The span structures have joints along the upper and lower straps of the trusses, have supporting portals and lightweight vertical joints, arranged in steps up to 6 m and are presented as angular junction plates in the joints of the abutment of the capping beams to the uprights of trusses. Sometimes, rigid unitized diaphragms of the flooring and capping structures of the gallery are used as joints on upper or on the lower straps of the trusses.

Flat frame supports of galleries have predominantly right angled forms, and are used also as the supports of trapezoidal and complex forms. In the latter case the upper part of the support usually is right angled, and the lower part, trapezoidal in outline.

The grating of the supports are usually taken out of consideration of their convenient transport with the least shipping cost. The height of the supports of the galleries is taken predominantly up to 20 m, in a number of cases, up to 40 m. The width of the supports

at the base constitutes 1/6 to 1/4 of their height. The first value is taken with the supports of right angled outlines, the latter - with trapezoidal and complex supports. The supports of right angled outlines having a width up to 3.5-4 m are used in galleries with their height up to 18-20 m, and with a width of 6-8 m up to 35-40 m. The supports of trapeziform and complex forms is recommended for use with a ratio of the height of the support to its width equal to 5 and more.

Figure 238 gives diagrams of an elevated inclined and horizontal metallic conveyor gallery, but in Fig. 239, diagrams of the support 1, shown in Fig. 238. The remaining supports are analogous to support 1, besides support 2, which in view of the greater width of the inclined part of the gallery, and also the limitation of its overall size is characterized by the presence of a cantilever (Fig. 240). The cantilever is used for a device with one side inclined, and the other side, with horizontal trusses. The second inclined and horizontal trusses of the considered section are located in one plane; the corresponding subassembly is shown in Fig. 241. The run-of-mine supporting subassembly of the horizontal trusses, set on pendulum flat supports, is given in Fig. 242. The lower supports of the trusses of the inclined span of a gallery are fixed. The opposing extreme supports of the trusses of a gallery are the rolling type.

a)

b)

c)

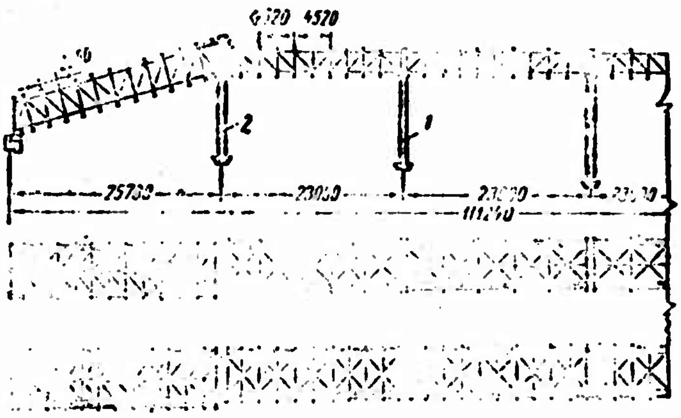

Fig. 238. Diagram of inclined and horizontal conveyor galleries: a) profile; b) plan for the upper belts; c) plan for the lower belts.

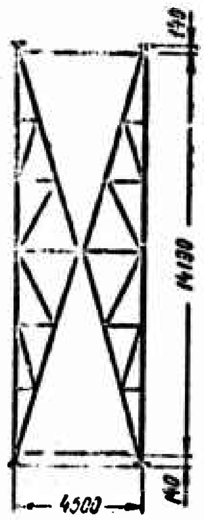

Fig. 239.

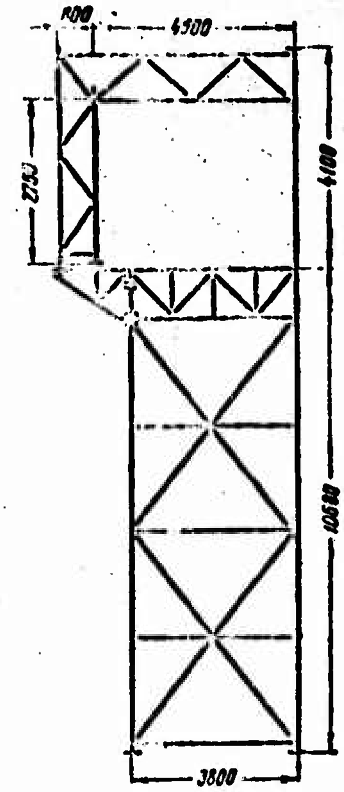

Fig. 240.

Fig. 239. The diagram of a flat support according to Fig. 238.

Fig. 240. A diagram of the support on the junction of the inclined and horizontal sections of the gallery according to Fig. 238.

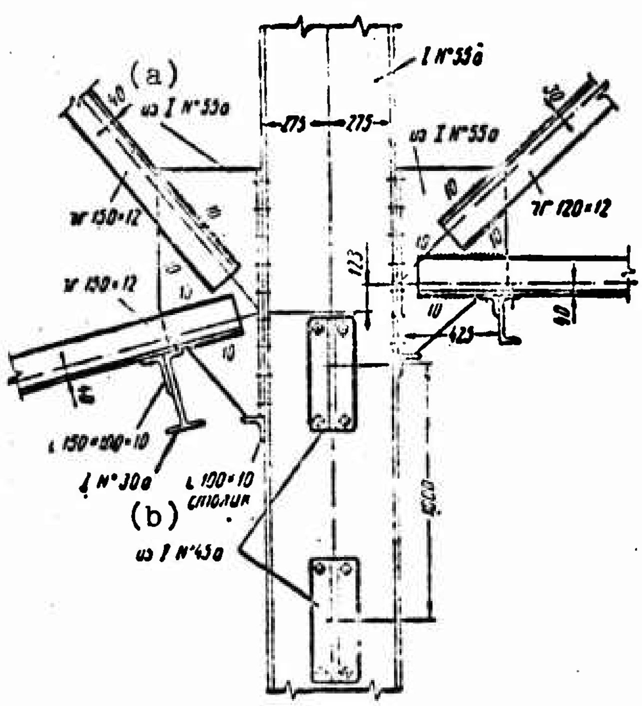

Fig. 241. Subassembly at the junction of the inclined and horizontal sections of the gallery.

KEY: (a) made of; (b) plate.

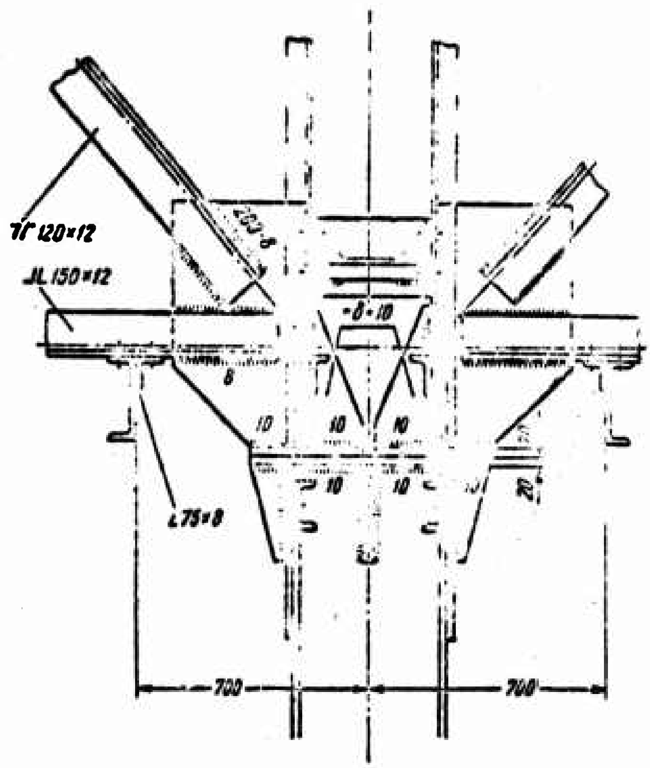

Fig. 242. The supporting joints of the horizontal trusses of the gallery.

Figure 243 give transverse cuts of the span structures of the galleries in horizontal and inclined sections.

Throughout the gallery there are warm enclosing designs. Slabs of portland cement fibrolite and mineral fiber serve as heat insulation. In the cappings and flooring the heat insulation is palced on the unitized sectional ferroconcrete slabs. In the walls the heat insulation is inserted in the composition of the sectional shields of the wall guard barrier of the galleries, faced on two sides with asbestos-cement corrugated sheets of a reinforced profile.

The overall length of the construction is 111.2 m, the expenditure of steel in gallery constitutes about 100 t, the average weight of the steel structure per 1 m of construction - about 0.9 t. In this case the weight of the span structure having an average width of 4.7 m constitutes 0.7 t to 1 m, or 0.145 t per 1 m^2 area of horizontal projection of the floor of the gallery. When evaluating these indexes one ought to consider the increased height of the gallery in its

inclined section, and the utilization here of the existing trusses and the considerable difficulties of joining the inclined and horizontal parts of the construction (see Fig. 240).

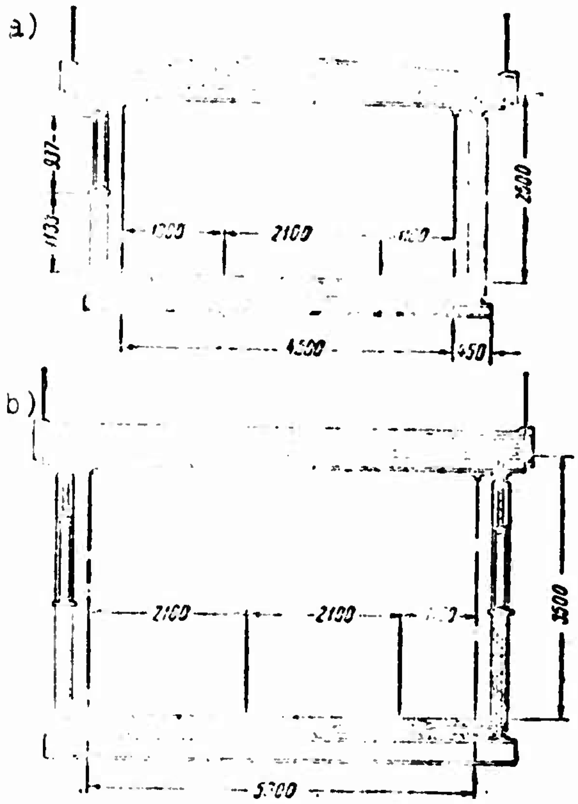

Fig. 243. A section of the galleries according to Fig. 238.

Loads on the span structures in this case are somewhat increased in connection with the utilization of the ferroconcrete ribbed capping slabs having a length of 4.5 m. At the same time the increase in the length of the sectional slabs of the basic sections of the gallery up to 4.5 m is reduced to the simplification of transverse beams and joints of the capping, and to the reduction and simplification of the assembly operations. The installation of the gallery was performed in short order despite the intersection of passages of the gallery by a number of existing routes, buildings and construction.

It is possible to note that with less width of the construction (3.0-3.5 m) such galleries are characterized by an expenditure of steel structures, equal to 0.15-0.16 t/m^2. In all the galleries the utilization of the increased sectional ferroconcrete capping slabs

based on sizes did not lead to an increase in the expenditure of the metal. Therefore, the design of the galleries, the span structures of which are characterized by a length, a multiple length of the standarized ferroconcrete and prestressed large-size capping plates, merit attention.

The utilization of the standarized large-size capping and flooring slabs and of the galleries is characterized by the diagram of the formation of the span structures of the galleries, which is as follows. The modulus of the length of the gallery, measured along the axis of the span structure, independent of the slope of the galleries is taken equal to 6 m. The lengths of the span structures are equal, in this way, to 12, 18, 24 and 30 m. Each of the named spans differs from the adjacent in presence or in the absence of the six-meter section, which has two panels (Fig. 244a).

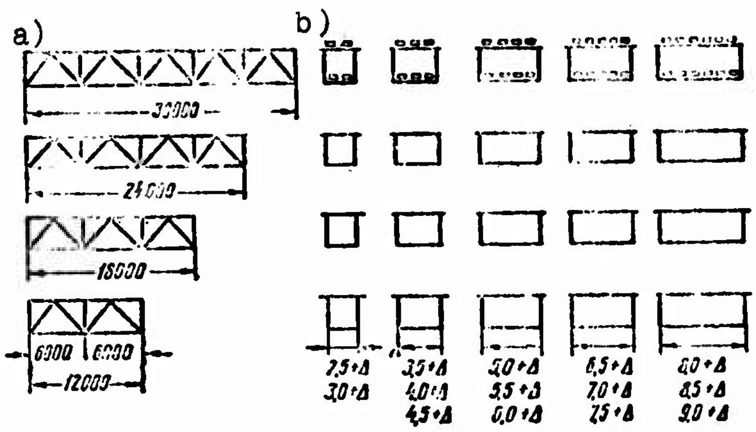

Fig. 244. Diagrams of universal conveyor galleries, composed of trusses with repeating sections 6 m in length.

Large size slabs on the beams, located in the joints of trusses can serve as the supporting designs of guards in a part of the capping and flooring of such a gallery. A series of floorings of the galleries can be formed with one type and dimension of slabs; for example, slabs with nominal sizes 1.5 × 6.0 m. In this instance with the width of the gallery in the axes of the trusses, somewhat larger than 3 m, two large size slabs (Fig. 245), with a width of 4.5 m - three slabs,

and with a width, somewhat larger than 6.0 m, - four large-size slabs are placed in the transverse section of the gallery. With the introduction of two type and dimension slabs, 1.5 × 6.0 m, and 1.0 × 6.0 m, it is possible to obtain a series of galleries with the width in the axes of the trusses, somewhat larger than 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, ..., m (Fig. 244b).

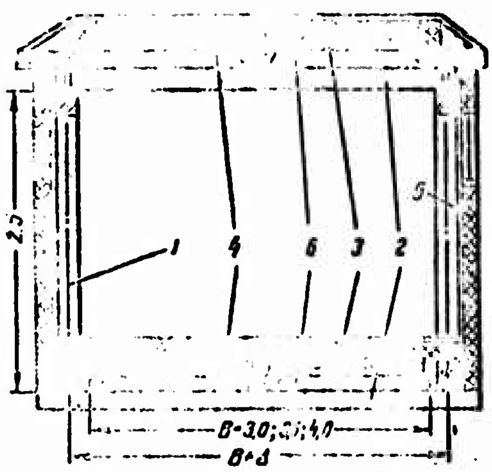

Fig. 245. Universal sectional conveyor gallery with ferroconcrete prestressed or steel trusses having a span of 12, 18, 24, 30, ..., m: 1 - truss from prestressed reinforced concrete or steel; 2 - transverse steel beam-junctions, spaced 6 m apart; 3 - structural standarized (and special) prestressed ferroconcrete capping and flooring slabs with a length of 6 m; 4 - heat insulation; 5 - the shields or panels of the wall guard; 6 - space for pipe lines and cables.

It is more expedient, however, to use two type and dimension slabs, 2.0 × 6 and 1.5 × 6 m, the presence of which makes it possible to form a series of galleries with a width in the axes of trusses, somewhat larger than 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, ..., m.

Slabs, 2 × 6 m, are characterized by a small expenditure of reinforced concrete, consisting of about 5.8 cm. If we also take into account that a series of slabs having a size, 1.5 × 6 m, can be replaced (in galleries of the width somewhat larger than 3, 4.5, 5. 6, 6.5, 7, 7.5, 8, 9 m) by more effective standarized slabs, 3 × 6 m, with a given expenditure of reinforced concrete of 5.2 cm; then, the overall expenditure of reinforced concrete on the capping and flooring of an area in the axes of the trusses will amount to about 0.1 m^3/m^2.

Among the mentioned structural slabs, only the slab, 2 × 6 m, is special for galleries; all the remaining slabs (1.5 × 6 and 3.0 × 6 m) are included in composition of standarized construction structures.

In conclusion it is possible to note the following:

as a result of using one special type and dimension of ferroconcrete ribbed slabs (2 × 6 m) with metallic trusses and transverse beams it is possible to form a series of span structures of conveyor galleries with spans 12, 18, 24, 30 m and with their width, 3, 3.5, 4, 4.5, 5, 5.5, 6, ..., m;

as a result of using two-three type and dimension ferroconcrete prestressed trusses with parallel straps (see Fig. 245) it is possible to form a series of span structures of sectional ferroconcrete conveyor galleries with spans 18, 24, 30 m with their width 3, 3.5, 4.5, 5, 5.5, ..., m.

Both with steel, and ferroconcrete prestressed trusses in the design of the described universal galleries it is expedient to maintain the metallic transverse beams, spaced at a distance of 6 m. The expenditure of metal to the transverse capping and flooring beams comprises for galleries having a width of 3 m, only about 0.01 t/m^2, and for galleries of a width of 6 m - about 0.02 t/m^2. The presence of steel transverse beams makes it possible to easily change the width of galleries over wide limits. Transverse beams are convenient and reliable elements of the joints on the upper and lower straps of the trusses.

Since the capping and flooring slabs are welded on the transverse beams and straps, unitized and represent invariable diaphragms, then there is no need to use steel joints on the upper and lower straps of the trusses.

The gallery according to Figs. 244 and 245 differs by the small number of transverse capping and flooring beams (2-3 times less than

in comparison with the above described solution). With a span of 24 m the number of intermediate transverse beams of a span structure are equal to 6 as opposed to 22 beams, used, for example, in a conventional inclined gallery. During the transition from small size ferroconcrete capping and flooring slabs to large size slabs the expenditure of reinforced concrete per unit of area of the gallery increase somewhat. However, the positive aspects of using large-panel slabs are so obvious, that the noted circumstance has no significance. Furthermore, because of the decrease in a number and weight of transverse beams and exclusion of joints the general weight of the steel structures can be substantially less in comparison with their weight for galleries with small-size slabs.

Figure 246 shows a diagram of the conveyor gallery, developed partially in accordance with the given positions. The complete length of the gallery is 246.8 m; in it there are two expansion blocks for 5 spans with the sizes of each, 24.4 m, in the axes of uprights and 24 m in the axes of the supports of the trusses. From the external side of each block fixed support are placed; all the remaining supports - mobile and presented as flat pendulum uprights. The transverse section of the span structure of a gallery is presented in Fig. 247. The gallery is composed of trusses having a span of 24 m, having transverse beams, spaced 6 m apart, lengthwise to the construction, and having steel joints on the upper and lower straps of the trusses.

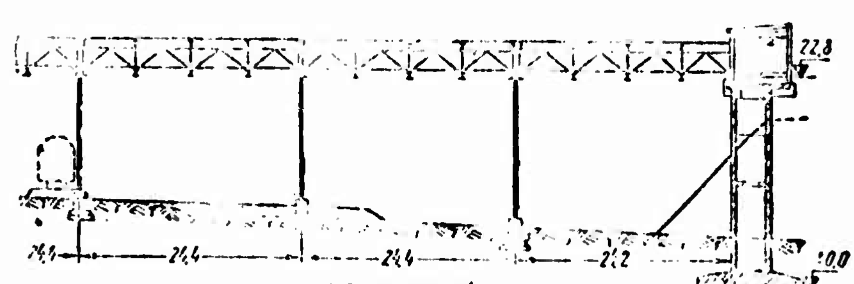

Fig. 246. Diagram of a conveyor gallery with trusses having a span of 24 m and sectional ferroconcrete capping and flooring slabs with nominal lengths of 6 m.

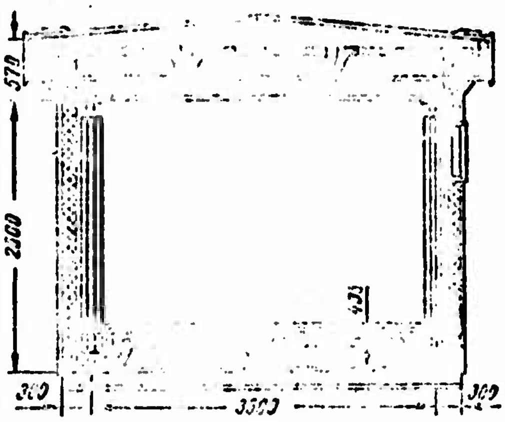

Fig. 247. The transverse section of the gallery with large-size capping and flooring slabs.

Standardized capping slabs of industrial buildings with nominal sizes, 1.5 × 6.0 m are used as capping and flooring slabs of the gallery. These slabs rest on the transverse beams and are braced to them using welding of matching parts. The wall guard of the gallery is represented by warm shields made of mineralized felt, set between two layers of asbestos-cement corrugated slabs of a reinforced profile.

The expenditure of the steel designs for the manufacture of the gallery is equal to 170 t, including 0.53 t/m of gallery or 0.15 t/m^2 of the area for the span structures. The indexes of expenditure of the steel for the given gallery are somewhat lower than for a metallic gallery with small-size slabs. However, a gallery with large-size slabs is characterized by a small number of elements of span structures and by the small number of more reliable bracings, respectively. On the basis of this when selecting span structures of galleries, one ought to prefer a design with large-size standarized capping and flooring slabs. The weight and indexes of the expenditure of steel according to the designs of universal galleries can be improved because of the lightweight capping designs using asbestos-cement shields and panels.

Independent of this, under the condition of using standarized ferroconcrete slabs it is necessary to show the possibility of reducing the weight of the steel structures of galleries by comparison with indexes given in the description of structures in Fig. 247. In

the gallery (see Fig. 247) with the limitedness of the assortment of rolled stock the following excesses are introduced in comparison with the recommended solutions of universal galleries according to Fig. 245:

1) the number of large-size capping slabs of the galleries is increased as opposed to that required by 1.5 times;

2) all the capping and flooring slabs of a gallery are only taken at a width of 1.5 m with the given expenditure of reinforced concrete at 6.3 cm;

3) the volume of insulation of the capping substantially increases as opposed to that required because of the excessive increased incline of the roofing;

4) steel joints are installed along the upper and lower straps of the trusses;

5) temporary loads were increased as opposed to the data, by 30%.

At the exception of the enumerated deviations one should expect a reduction in the weight of the steel structures for the examined gallery having a width of 3.5 m, by approximately 0.04-0.05 t/m^2, i.e., up to 0.1-0.11 t/m^2.

Standard metallic conveyor galleries of a width of 4.5, 5.0 and 5.5 m have been developed for spans of 18, 24 and 30 m. The galleries are designed for two conveyors with a width of the belts:

at gallery width of 4.5 m - 800 and 800; 650 and 1000 mm;
" " " " 5.0 m - 1000 and 1000; 1200 and 800; 650 and 1400 mm;
" " " " 5.5 m - 1200 and 1200; 1000 and 1400 mm

A distance between the axes of the trusses of the galleries corresponds to the width of galleries (4.5, 5.0, 5.5 m). The lengths

of the trusses are equal 18, 24 and 30 m. The trusses have a delta-shaped grating with the uprights and suspensions; the length of the panel is 3 m, the height of all trusses is 2.6 m. The span structures do not change with a change in the angle of a slope from 0 to 18°, the change in the incline is reflected only in the design of the supporting joint of the span structures.

The supports and foundations in the composition of standard galleries are not included and are developed in each case as individual designs, depending on the area relief, the conditions of approximity to buildings and construction, the nature of soils and so forth.

The galleries are either heated or unheated. The heated galleries have lightweight heat insulation for the walls, cappings and floorings. The heat insulation of the roofing and flooring is made from autoclave foam concrete slabs with a volumetric weight of 400-500 kg/m^3. The enclosing designs of walls based on one variant are arranged using warmed asbestos-cement shields with a wooden frame with heat insulation made of mineralized slabs of volumetric weight of 300 kg/m^3. In another variant for the barrier of the walls single-layer panels made from reinforced autoclave cellular concrete (Fig. 248) is used.

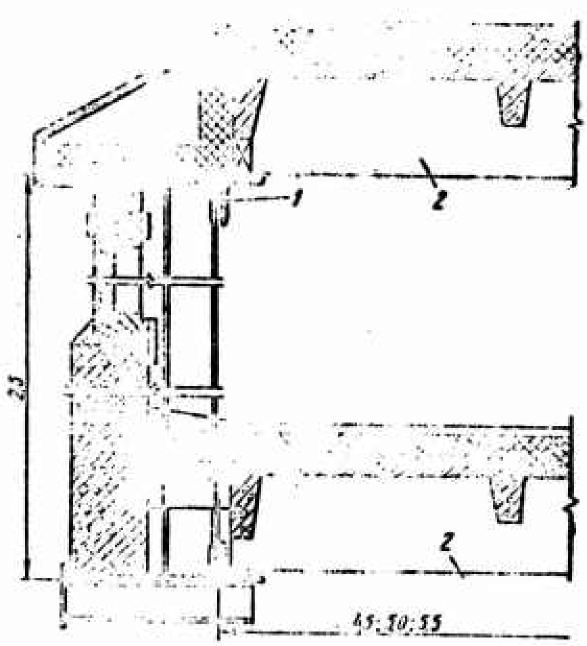

Fig. 248. Standard metallic conveyor galleries with a span of 18, 24 and 30 m; section; 1 - trusses with height of 2.6 m along the pad eyes; 2 - sectional ferroconcrete of prestressed capping and flooring slabs of sizes, 2.98 × 4.44; 2.98 × 4.94 and 2.98 × 5.44 m.

Special large-size ribbed prestressed ferroconcrete slabs, 3×4.44 m in size rest on the upper and lower chords of the steel trusses for a gallery of a width of 4.5 m, 3×4.94 m for a gallery of a width of 5.0 m, and 3×5.44 m for a gallery of a width of 5.5 m (Fig. 248). It is obvious that at each width of the conveyor gallery a specifically designed structure of prestressed capping and flooring slabs should also be developed, which should have a length, which corresponds to the width of the gallery (3.44, 3.94, 6.44, 6.94, 7.44, 7.94 m, etc.). This is the great deficiency of the described standard metallic galleries.

It is necessary to show only that the direction of the spans of slabs by design should be changed so that the slabs lay not across, but parallel to the trusses of the standard galleries. In this instance the chief deficiency of standard galleries diminishes, and all the capping and flooring slabs can have an identical length, equal, for example, to 6 m with various spans (18, 24, 30 m) and at different width of the galleries, which consist not only of 4.5, 5.5 m, but also 3, 3.5, 4.0, ..., 6.0, 6.5, 7.0, 7.5, ..., m.

The transverse slabs of standard galleries are welded on the matching parts to the flanges of the corners of the upper and lower chords of beams; the seams between the slabs are unitized. In accordance with the presence of obtained diaphragm in this way the steel joints on the upper and lower chords of the trusses are excluded. On the supports of the span structures steel portals are provided.

In the lower joints of trusses fixed supports, and in upper extreme joints - movable supports are provided. In the presence of intermediate pendulum uprights, the support of the trusses, plane linear using narrow ribs, the supporting surface of which is planed in accordance with the incline of the gallery.

In accordance with the exclusion of steel joints, beams and the utilization of the prestressed ferroconcrete capping and flooring

slabs having a width of 3 m (with a given expenditure of reinforced concrete of about 5.5 cm) the metal structure galleries are characterized by good weight indexes.

For heated galleries with spans of 24–30 m and with the width of galleries at 4.5 m and with a snow load of 150 kg/m^2, the weight of metal structures constitutes 0.08-0.11 t/m^2. The weight of the metal structures for the heated galleries having a width of 5.0 m is less by 8-9%, and for galleries of a width of 5.5 m - up to 10-11%. The weight of the metal structures of the heated galleries of the same design with a span of 24-30 m, but with a width of 3.0-3.5 m, will be 0.09-0.13 t/m^2. The weight of the metal structures of heated galleries with a span of 18 m constitutes 75-80% of the lesser above mentioned values (which correspond to galleries with a span of 24 m).

The weight of the metal structures of span structures of heated galleries with spans of 18-24 m constitutes 90-95% of the weight of the metal structures of heated galleries. The lesser one presented here are the values percaining to galleries having a width of 4.5 m, larger ones - galleries of a width of 5.0 and 5.5 m. With spans of 30 m and a width of 4.5 m, the weight of the metal structures of unheated galleries constitutes 75-80%, but with the width 5 and 5.5 m - 85-90% of the weight of the metal structures of heated galleries.

The weight of the steel of the ferroconcrete slabs above, in all cases, are shown.

The difference between the indexes of the weight of the metal structures described above of universal (Figs. 244 and 245) and standard galleries given here (Fig. 248) in all cases will be distinguished by the amount of weight of the transverse beam-joints, which comprise about 0.01 t/m^2 for galleries having a width of 3 m, and about 0.02 t/m^2 of the area of the gallery for galleries of a width of 6 m.

Ferroconcrete conveyor gallereis. There are the examples of the erection of ferroconcrete prestressed galleries with span structures made from channeled ferroconcrete sections with spans of 36 m (Fig. 249) and 84.7 m.

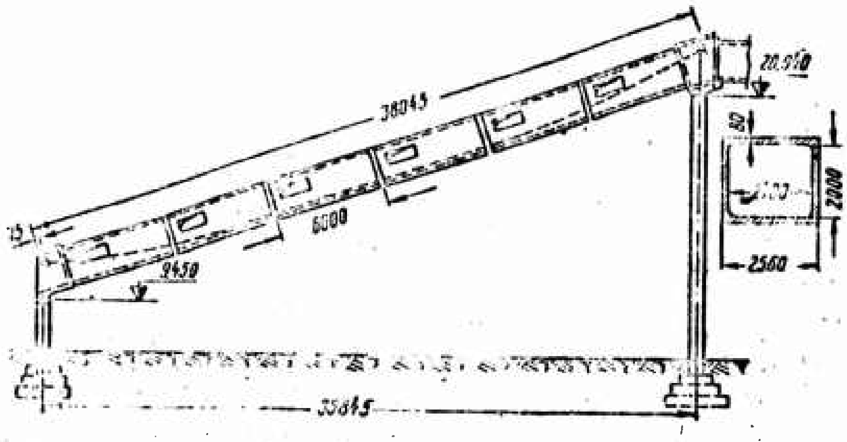

Fig. 249. A conveyor gallery with a span of 35.8 m using stressed-reinforced span structure, assembled from channeled ferro-concrete sections.

Standard span structures of galleries made from channeled ferro-concrete units have been developed. The width of these galleries is 2.5, 3.0 and 3.5 m; the spans of the galleries having a width of 2.5 and 3 m - within the limits of 27-51 m, and galleries 3.5 m - from 12 to 45 m.

The presentation of galleries with span structures made from channeled units gives a description below of galleries of a width of 3 m. The span structure of the gallery consists of intermediate sections and supporting portals. Each intermediate channeled section consists of two separately made ribbed walls and two flooring plates, which unite as a box during assembly. The sizes of the intermediate channeled section, 3.2 × 2.6 × 5.85 m; the seams between sections have a width of 0.15 m, and the nominal length of a section is 6 m. The thickness of the slabs of intermediate sections is 80-90 mm. The supporting portals have a length of 1.5 m and present monolithic boxes made of reinforced concrete with the thickness of the walls of

270-350 mm. The intermediate channeled sections and supporting portals are united by means of weldings with subsequent cemented transverse seams. For the manufacture of the sections and portals concrete of a grade 400 is used, for the filling of the seams - concrete of grade 500.

The high-strength stressed steel framework of the span structure consists of beams of high-strength carbon wire having a diameter of 5 mm with an ultimate strength of 17,000 kg/cm^2. The steel framework of the beams is located inside the gallery along the walls, and in supporting protals - inside the apertures in the widened walls of the portals. The axes of the beams form curves, close to that of parabolas. The fastening of the beams to intermediate sections is secured by matching parts.

The heat insulation of the walls, capping and flooring of the gallery is done with cellular concrete having a thickness of 100 mm.

The weight of the ferroconcrete designs and heat insulation of the span structure of the gallery having a width of 3 m with spans of 27-51 m constitutes 1.45-1.50 t/m^2 of the area of the gallery. The expenditure of the reinforced concrete is equal to 0.49-0.52 m^3 per 1 m^2 of area. The expenditure of steel for the same span structures with a span of 27 m is characterized by the value of 0.07 t/m^2 of the area of the gallery. In this case about 0.01 t/m^2 of high-strength wire is necessary. The given expenditure of metal in this case is equal to 0.08 t/m^2. One ought to show that the standard metallic conveyor galleries (Fig. 248) having a width of 4.5 m with a span of 24 m are characterized by an expenditure of metal of 0.07-0.08 t/m^2, and the metallic galleries of the same type with a width of 3 m - by an expenditure of metal of about 0.09 t/m^2. The expenditure of steel for the span structures of ferroconcrete galleries described here with channeled sections along with a span of 51 m is equal to 0.09 t/m^2, of which almost 0.03 t/m^2 is attributed to high-strength wire; the given expenditure of metal in this case is 0.14 t/m^2. The weight of the steel structures of standard metallic galleries (see Fig. 248) with a span of 30 m and a width of 4.5 m is equal to

0.11 t/m^2, but the weight of the metal of such galleries with their width at 3 m constitutes about 0.13 t/m^2.

It is possible to note that the indexes of the expenditure of reinforced concrete and of steel for galleries with channeled sections with their width at 2.5 and 3.5 m are somewhat improved as opposed to indexes for galleries of a width of 3 m. On the whole, however, galleries with channeled sections are characterized by excessively large weight, by high expenditures of steel and of concrete of high grade and by a considerable input labor.

Figure 250 shows a sectional ferroconcrete conveyor gallery of an ore-concentration combine. This gallery, for the greater part is marked out with a small incline, which approximately corresponds to the surface. The average height of the gallery is about 7 m, with a width of 6 m, and the length of the construction is 2063 m. The spans of the gallery on the incline are equal to 12 m. The distances between the sectional ferroconcrete uprights of the construction are also close to 12 m (the small section of the gallery has uprights, spaced at a distance of about 6 m). The expansion blocks of the galleries have a length of 60 m and are limited by uprights. The passage from the central upright to the span structure of the gallery is provided for because of the utilization of transverse double-cantilever sectional ferroconcrete prestressed cross members, on which the main longitudinal beams of the span structure rest directly. Sectional ferroconcrete prestressed standarized frame-supporting beams (PBN4 and PBN4k) with an inclined lower chord having a length of 11.94 m are used as the main beams. The ribbed flooring slabs rest directly on the main beams and the trasverse ferroconcrete frames are set at a distance of 6 m. Single-layer wall panels made of cellular concrete are braced to the uprights of the frames and the ferroconcrete capping slabs rest on the crosspieces of the frames.

The expenditure of the reinforced concrete of grades 400 and 200 on the upper structure of the gallery, the main beams and transverse Π-shaped frames capping and flooring slabs constitute 0.36 m^3

per 1 m^2 of area of the gallery. For columns, crossmembers and foundations the additional expenditure of reinforced concrete constitutes 0.22 m^3 per 1 m^2 of area. The expenditure of steel on the span structures - about 0.10 t/m^2 of the area of the gallery, which exceeds the expenditure of steel, necessary for a standard metallic conveyor gallery having a width of 5.5 m with spans of 18 m (see Fig. 248), which constitutes about 0.07 t/m^2. The weight of the span structure of the gallery constitutes about 1.5 t per 1 m^2 of area.

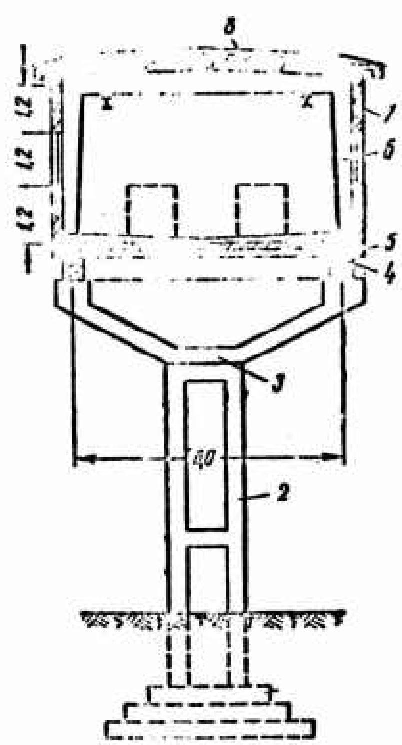

Fig. 250. Structural conveyor galleries with a span of 12 m made from prestressed reinforced concrete: 1 - foundation; 2 - structural ferroconcrete upright; 3 - structural prestressed ferroconcrete crossmember; 4 - structural ferroconcrete prestressed (standarized frame-supporting) beam with an inclined lower chord having a length of 11.94 m; 5 - structural ferroconcrete flooring slabs; 5 - structural ferroconcrete transverse frame; 7 - structural single-layer panels of the walls; 8 - structural prestressed ferroconcrete capping slabs.

The general indexes for the gallery can be somewhat improved, specifically because of a more rational solution of the lower structure With a considerable width of the gallery and comparatively small height the introduction of heavy prestressed crossmembers is not a rational measure. During the arrangement of single columns under

the main beams, i.e., using two simple columns spaced 6 m across, and with the introduction of lightweight transverse joints of the columns, the indexes of the expenditure of reinforced concrete are reduced. However, the examined ferroconcrete structural gallery, despite its limited span (12 m) and height, is characterized, on the whole, by the increased expenditures of the basic construction material, steel and exceedingly large weight of the span structure.

The satisfactory gravimetric and other technical-economical indexes of the conveyor galleries, carried out during the preferred utilization of supporting ferroconcrete structures, can be obtained only under the condition of a considerable decrease in the weight of the gallery. This mission, in turn, solves itself only under the condition of using ferroconcrete, specifically prestressed, support designs at a minimum necessary quantity. The weight of the enclosing designs, primarily the weight of the walls and capping of the gallery, usually also produces a considerable load on the support structures of the gallery and it should be, in all cases, as short as possible. If the utilization of the rock, brick and large block masonry of the walls in the galleries is inadmissible, then the utilization of wall panels of relatively increased weight and single-layer panels made from reinforced cellular concrete is not recommended. The weight of the barriers of the walls and the cappings of the galleries, including the weight of heat insulation should be limited by the value of 0.10-0.15 t/m^2, which determines the extensive use of asbestos-cement of shield and panels for enclosing designs of the galleries.

Figure 251 shows the gallery, whose spans are equal predominantly to 12-18 m, and sometimes 6, 9, 15 m. The basic design of the span structure of the gallery is, in general, a special large-size ferroconcrete prestressed ribbed slab 1. In the necessary cases this slab can consist by length, of units in size, for example, 6 m, joined in one general design by means of tension rods or a bundle of reinforcement rods. A ribbed slab can also consist of two beams and transverse slabs, united into a single general design with the help of welding and cementing of the seams. Lightweight transverse frames 2, are set

directly on the main ribs of a plate, and made with heated galleries from aluminum alloys or steel, and with unheated galleries, in a number of cases, also using wood.

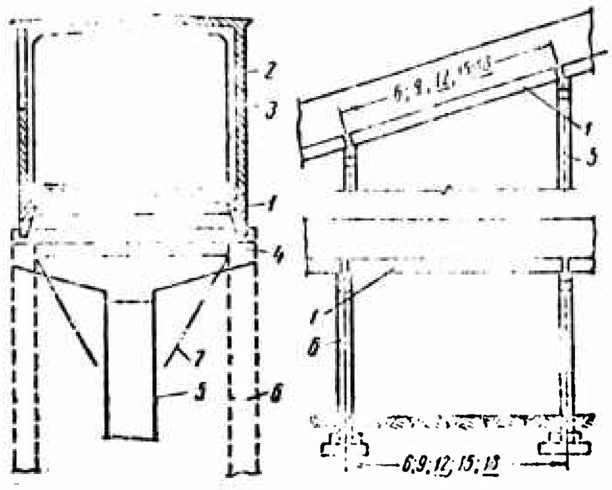

Fig. 251. Structural ferroconcrete prestressed conveyor gallery with a span of 6, 9, 12, 15, 18 m: 1 - prestressed ferroconcrete ribbed slab; 2 - lightweight transverse frame made from steel, aluminum or wood; 3 - asbestos-cement shields or panels; 4 - structural cross-members; 5 - upright of the support for its central arrangement; 6 - upright of the frame; 7 - joints.

Asbestos-cement sheathing or paneling 3, are the barriers and heat insulation of the gallery, and depend on a distance between the frames, are braced directly to crosspieces and uprights of the frame, and sometimes to the capping shafts and to the joint-spreaders of the frames. Using transverse frames every 3 m the utilization of hollow, heated asbestos-cement slabs is possible, the fastening of which can be done also directly to crosspieces and the uprights of the transverse frames.

With spans of the gallery at a slope of 12 m (and also at 6 m) one could use standarized designs, specifically with lightweight galleries having a width of 3 m the utilization of large-size ferro-concrete prestressed capping slabs with nominal sizes, 3 × 12 m, along with an increase in the steel framework in the main ribs of the slabs, is possible. In this instance with the width of the gallery at 3 m, height of 2.2 m and with barriers made from asbestos-cement hollow

slabs, the calculated load on the ferroconcrete ribbed slab 1 (Fig. 251) during its calculation of the main rib should not exceed 1.3 t/m or 0.9 t/m^2 of the area of the slab. The given expenditure of reinforced concrete per unit of area of the span structure of this gallery constitutes about 0.07 m^3. The overall expenditure of steel, necessary for the reinforcing of the structural flooring slabs and for the manufacture of the lightweight steel transverse frames, should not exceed 0.03 t/m^2. With wooden transverse frames the expenditure of steel is about 0.015 t/m^2. The weight of the span structure of the gallery is 0.6 t per 1 m^2 of area of the gallery.

In the case of the utilization of standarized structural ferroconcrete large-size capping slabs with nominal sizes, 1.5 × 12 m for flooring of the gallery having a width of 3 m the expenditure of reinforced concrete increases to 0.09 m^3 per 1 m^2 of area with an increase in the weight of the gallery up to 0.7 t/m^2. In this case the reinforcement of the steel framework of the main rib of the slabs is also necessary.

On the basis of the given data it may be concluded that under conditions of utilizing special (but sometimes standarized) large-size ferroconcrete prestressed ribbed slabs having spans of 12-18 m and at lightweight barriers the erection of the simplest, lightest and most expedient span structures of conveyor galleries (Fig. 251) is possible. The unit weight of these span structures is two times less the appropriate weight of the span structures described above for ferroconcrete conveyor galleries (see Figs. 249 and 250). The expenditure of reinforced concrete per 1 m^2 of area of the gallery with ribbed prestressed slabs (see Fig. 251) is 4 times less, the expenditure of steel is 2-3 times than the corresponding indexes for the span structures of the above described ferroconcrete conveyor galleries.

The use of single ferroconcrete uprights 5 with crossmembers (see Fig. 251) is expedient for relatively narrow and lightweight galleries with moderate response to the action of the wind. For wide galleries frequently it is more expedient to use two uprights 6 with

the necessary joints 7. The selection of the type of uprights of the gallery is determined by a comparison of the variants.

With ferroconcrete designs of the galleries with spans of 18, 24, 30 m under usual conditions the utilization of universal conveyor galleries with prestressed ferroconcrete trusses (see Fig. 245) are most expedient. These special trusses with parallel chords can have constant height, consisting of 2.5 m for the various spans shown here based on the external overall sizes. The transverse beam-joints with ferroconcrete trusses for expediency can be metallic. The width of the galleries in the presence of one special type and dimension of ferroconcrete prestressed capping and flooring slabs changes, as shown above, every 0.5 m (beginning from a width of 3 m).

With an average span of 24 m and width of the gallery of 3.5-6.0 m the span structure of the universal conveyor gallery with prestressed ferroconcrete trusses is characterized by a weight of 0.9-0.7 t per 1 m^2 of area of the gallery, which is only somewhat higher than the weight of the conveyor galleries with metallic trusses and almost 2 times less than the weight of the span structures of galleries with ferroconcrete channeled sections. The overall expenditure of reinforced concrete for trusses and capping and flooring slabs of a span structure constitutes 0.13-0.14 m^3 per 1 m^2 of area, the expenditure of steel, including the steel for transverse beam-joints (see Fig. 245) and the steel framework of the capping and flooring slabs of a span structure, constitutes 0.035-0.05 t per 1 m^2 of area of the span structure of the gallery. The given indexes of the expenditure of the reinforced concrete is 3 times less, and of steel 2 times less than the corresponding indexes of the expenditure of reinforced concrete and steel of span structures made from channeled ferroconcrete sections, and also from other ferroconcrete conveyor galleries.

Conveyor galleries with wooden and combined supporting structures. Frequently galleries with imes with a relatively short period of operation are made of wooden structures.

Figure 252 shows the simplest wooden conveyor gallery, arranged between two fire-proof zones. The supports of the gallery are presented with two transverse frames 1, set on concrete foundations 2. The shafts, arranged alternately rest on the crosspieces. The upper crosspieces of the transverse frames can be presented as double tongs or as single adapters. In the latter case large sections of wood are necessary, but the support of the shafts is more expedient. For the improvement of the conditions of the support of the shafts in the described case continuous padding is provided for the tongs of the crosspieces of a frame.

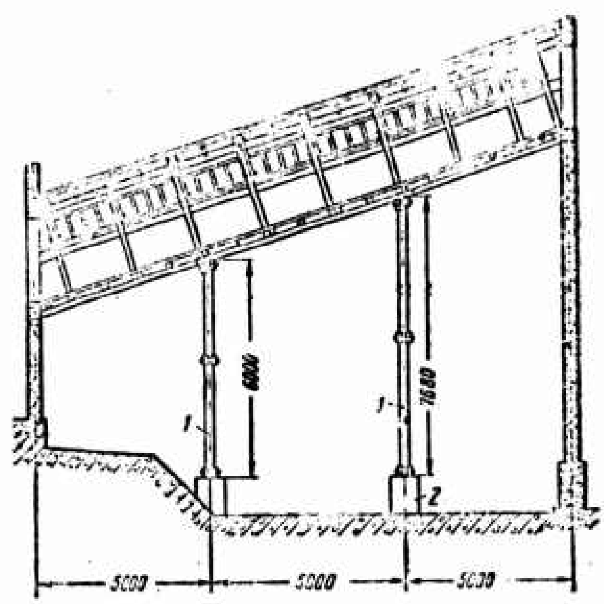

Fig. 252. A wooden conveyor gallery.

The shelter of the gallery is cold and is presented with lightweight transverse frames made from planks and nails. The frames are set at right angles to the axis of the gallery every 2 m over its length. In the necessary cases joints are attached to the transverse frames which assure the stability in a longitudinal direction. The barrier of the walls of the gallery is represented by sheathings of asbestos-cement corrugated sheets. Sometimes they are used as plank sheathing. The single over-suspended sashes are installed from one side of the gallery. The rolls of roofing material used as a continuous protective layer and lathing, lies direct on the crosspieces of the frames. The flooring lies on the lightweight transverse

bracings, set lengthwise to the gallery at a distance up to 1.0 m from one another. The described gallery can be used for more than 20 years.

Figure 253 shows a conveyor gallery with wooden trusses and span structures. The walls of this warm gallery (Fig. 254) are composed of uprights 140 mm thick. The described design of the walls of a gallery under dry air conditions are used in the forested region. Under other conditions, from considerations of the economy of wood, and also for the purpose of control against possible rotting of the walls, the barriers of the warm galleries with wooden supporting structures are expediently used composed of the above described sheathing with their heated mineralized slabs or felt and sheathing with two sides of asbestos-cement corrugated sheets.

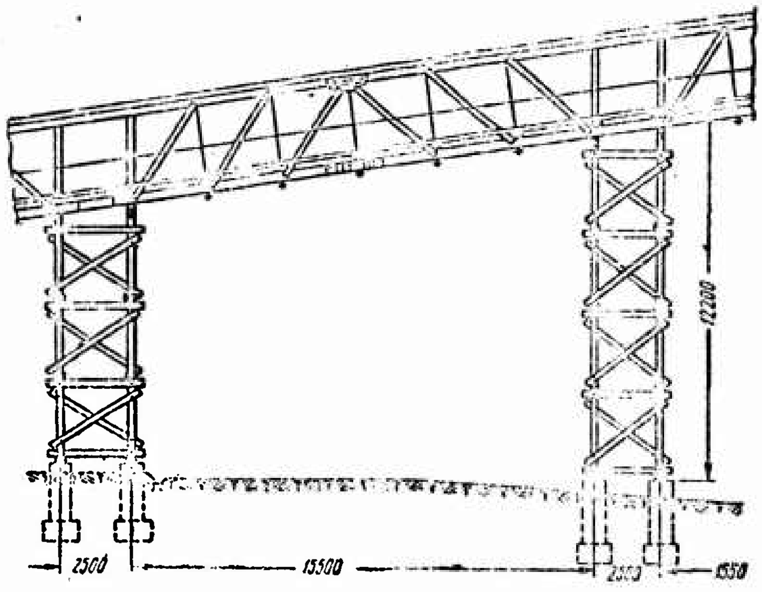

Fig. 253. Conveyor gallery with wooden trusses of the span structure.

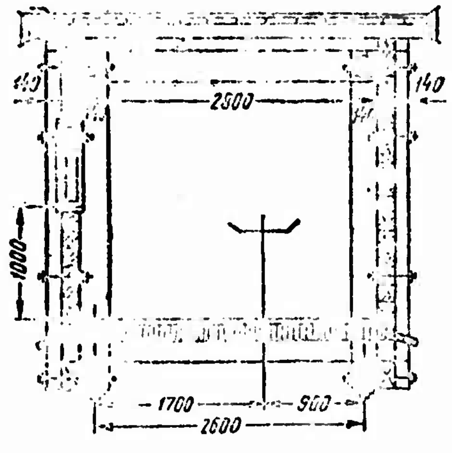

Fig. 254. Section of a gallery with wooden trusses (applies to Fig. 253).

Wooden conveyor galleries with spans at 30 m and with a usual width of 4 m are used. The span structures of these galleries are represented by wooden segmented trusses of increased rigidity having a height of 5.2 m, transverse beams, joints, supporting portals. The supporting structures are completely protected from the effect of moisture. Such span structures are characterized by sufficient rigidity and they are used on capital supports at the mines for relatively prolonged periods of operation. The utilization of such conveyor galleries usually has a place in ore storage bins.

With the erection of wooden conveyor galleries one ought to pay attention to protection from rotting, specifically the supporting frames, which should be sanitized. Furthermore, it is also expedient to provide protection to slightly inclined uprights of frames from the three external sides by three vertical stained planks on short padding, which cover simultaneously the supporting joints. The wood of the supporting structures thus does not undergo the action of moisture; on the other hand, the good ventilation of joints is provided for. With the limited number of supporting uprights and their spacing by design, sometimes the utilization of sheathing on the outer surface of the complex supports is more expedient than if there are more spans and more galleries.

The bottom of the wooden supports should be located at heights not less than 0.6-0.8 m over the level of planning. Less distances in a vertical line are inadmissible, since one ought to consider the usually observed spillage on the supports during the transport of the ore, rocks and tailings.

Lightweight ferroconcrete uprights can expediently be used as supports of the wooden span structures of the galleries. This design of the conveyor galleries in its simplicity is characterized by relative stability, by a considerable increase in the possible period of operation and by a substantial increase in the degree of fire-preventative safety.

With a further increase in the number of ferroconcrete elements in the composition of the galleries of a combined design, a solution of the gallery (Fig. 255) will be obtained, whereby the supports, beams and slabs are represented by ferroconcrete structures, supports of the conveyor; wooden enclosing designs are installed above.

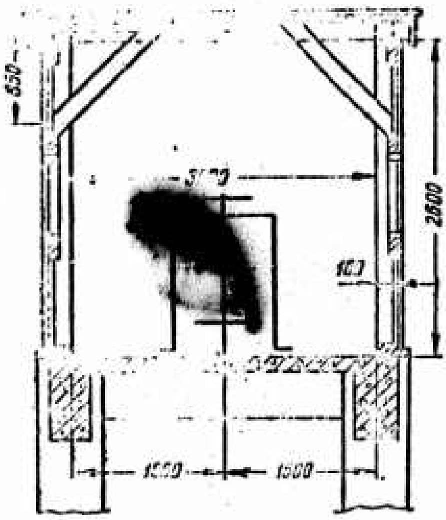

Fig. 255. Span structure of a conveyor gallery with a ferrococrete ribbed slab, with transverse frames and with a barrier of the walls made from asbestos-cement sheets

Sometimes with narrow galleries combined designs with span structures of the galleries, represented by a single ferroconcrete beam, T-shaped in section or by a single centrally set ferroconcrete prestressed beam, which lines on single columns, are also used. Lightweight wooden frames, similar to those in Fig. 255, which support usually unheated barriers of the walls and cappings of galleries, are set on the beam symmetrically to its axis.

The future development of the combined designs of galleries mentioned here are the above described conveyor galleries, which are characterized by a span structure using structural ferroconcrete prestressed ribbed slab with a span of 12-18 m (see Fig. 251). The shelter of the conveyor belt in this instance is provided by the lightweight metallic or wooden transverse frames supporting shields and panels of the barriers of the walls and capping of the gallery placed on the ribs of the slab.

3. The Calculation of the Galleries

The overall loads of the conveyor galleries are taken according to the Construction Standards and Rules, allowing for the coefficients of an overload.

A normative useful load in the passages of the galleries, including the weight of running belt or grating, constitutes 200 kg/m^2 with a coefficient of overload of 1.3-1.2.

A normative load because of ore and water on the floor is taken within the limits of 50 kg/m^2 of a horizontal projection of the floor of the gallery with a coefficient of overload of 1.2.

A normative load from the weight of pipelines and cables is taken equal to 50 kg/m^2 of the horizontal projection of the ceiling of the gallery with a coefficient of overload of 1.2. This load is also arbitrary and frequently amounts to a linear local load at the walls over the passages of the galleries, where the padding of the separate pipelines and cables is most convenient and conventional.

The loads from the weight of conveyors with ore depend on the width of the belt and volumetric weight of the transported material. With a volumetric weight of material of 2.5 t/m^3, the corresponding weight of the largest part of iron ore, the weight of the conveyor with a load per 1 m of its length and per 1 m^2 of area of the frame constitutes:

at belt width of	500 mm	-	0.15 t/m	or	0.15 t/m^2
" " " "	650 mm	-	0.20 "	"	0.20 "
" " " "	800 mm	=	0.30 "	"	0.25 "
" " " "	1000 mm	-	0.40 "	"	0.27 "
" " " "	1200 mm	-	0.55 "	"	0.32 "
" " " "	1400 mm	-	0.70 "	"	0.36 "
" " " "	1600 mm	-	0.9 "	"	0.42 "

The dynamic effect of the conveyors can be evaluated approximately by the coefficient 1.05-1.2 (a larger value pertains to heavy

conveyors). Considering the certain conditionality of the load on the pipelines, its is possible to conclude the following.

With conveyors with belts having a width up to 1000 mm one cannot inclusively divide the calculation into sections, occupied by conveyors and passages, considering the effect of the weight of equipment, the weight of the ore, the prosor*, gratings, water and a working load, from the weight of the maintenance personnel, which correspond to the average normative load of the floor of the gallery, equal to 0.3 t/m^2 with an overall coefficient of overload of 1.3.

With conveyors having belts with a width of 1200 mm and more, one ought to divide the sections of galleries, occupied by the conveyor and by the passages. The normative loads in the passages in this instance can be considered equal to 0.2 t/m^2 with a coefficient of overload of 1.2-1.3. The loads under the conveyors with belts having a width of 1200-1600 mm are taken in a size of 0.55-0.9 t/m (per 1 m of length of the conveyor) with a coefficient of overload of 1.3-1.2, coefficient of dynamicity of 1.1-1.2 and an overall coefficient of overload (load) predominantly of 1.4. Lesser values of the partial coefficient of overloads, eoual to 1.2, are taken using accurate data about the weight of the equipment and ore.

Loads from a conveyor are taken along two lines of the supports of its frame. In the transverse section of the gallery these loads are presented by two concentrated forces, applied approximately on the outer side of the frame of the conveyor. The loads from prosor and pieplines during their separate assembling are determined, as shown above.

It is obvious that the separate collection of the loads of the gallery can be made even with conveyors having belts of a width up to

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*[Translator's note: the term "prosor" cannot be found in available sources].

1000 mm. One ought to consider in this instance the minimum value of the coefficients of dynamicity.

The flooring sections with installed driving stations of the conveyors are calculated according to the data about the weight of the installed equipment along with the introduction of coefficient of overload of 1.3, the arbitrary dynamic coefficient 1.5, and overall coefficient of overload of 2.0.

During the determination of the calculated loads from the weight of the designs and heat insulation the coefficients of overload of 1.1-1.2 and 1.2 are considered; during the determination of the effect of tension devices longitudinal tension of the conveyor belts a coefficient of overload of 1.3 is introduced.

The weight of the span structures of the galleries depend on accepted designs and it fluctuates within relatively large limits. The weight of the galleries, pertaining to 1 m^2 of area of the floor, is characterized by the following values:

1) with ferroconcrete beams with spans of 9-12 m, the ferroconcrete slabs of the span structure and brick walls with a thickness up to 380 mm for galleries of a width of 2.5-4 m, the weight is equal to 2.1-3.0 t;

2) the same, with brick walls of a thickness of 250 mm - 1.6-2.2 t;

3) with ferroconcrete prestressed beams with spans of 12-18 m, ferroconcrete slabs and brick walls having a thickness of 250 mm, the weight constitutes 1.5-2 t;

4) with ferroconcrete prestressed enclosed channeled units of span structures of galleries having a width of 3 m, warmed by cellular concrete with volumetric weight of 0.6 t/m^3, with spans of 27-51 m, the weight is 1.45-1.5 t;

5) with ferroconcrete prestressed beams with spans of 12-24 m, warmed by cellular concrete, with a width of the galleries of 2.5-4 m, the weight is 1.2-1.5 t;

6) with ferroconcrete prestressed trusses with spans of 18-24 m, with prestressed capped and flooring slabs, wall shields using asbestos-cement sheets - 0.7-0.9 t;

7) with ferroconcrete prestressed ribbed slabs of span structures with a span of 12 m along with lightweight shelter of the gallery - 0.6-0.9 t;

8) with steel trusses with spans of 18-30 m, ferroconcrete prestressed capping and flooring slabs, lightweight wall warming asbestos-cement shields - 0.6-0.8 t;

9) with steel trusses, ferroconcrete prestressed flooring slabs and asbestos-cement shields of walls and capping - 0.55-0.7 t;

10) with wooden designs, heated and unheated barriers - 0.2-0.4 t;

The weight of the span structures of conveyor galleries depends upon the number of reasons and it changes within extremely wide limits; therefore, one ought to specify the weight in the process of calculation. The data given here about the weight of galleries, which includes the weight of supporting designs, barriers and heat insulation can be used from the first determination of loads and the selection of the sections of the main slabs, beams and trusses of the span structures of the galleries, and also for a judgment about the loads on the supports and foundations.

During the calculation of the span structures, the main supporting designs of which are represented by ferroconcrete ribbed slabs, and also by ferroconcrete or metallic beams and trusses (with the loads towards the top) with horizontal joints at the level of the

upper chords of the trusses or towards the top of the beams, the horizontal windy loads are transmitted to the slab of the ribbed or on horizontal joints along the upper chord.

In the presence of the portal frames, and also of joints or diaphragms along the upper and lower chords of prestressed ferroconcrete or metallic trusses, the windy loads are usually distributed half-and-half between the joints along the upper and lower chords.

During the calculation of metallic or ferroconcrete supporting frames the effect of a local evenly distributed windy load usually is not considered.

The uprights of flat supporting span metallic frames of galleries with the value of their flexibility along the gallery equal to 60 and more, are considered in a calculated diagram as existing hinges at the base and top of the upright, i.e., as pendulum uprights, the top of which is displaced in a horizontal direction in accordance with a change in the length of the span structure with changes in its temperature.

The supports of the more rigid metallic uprights, both conventional monolithic or unitized at the base of the sectional ferroconcrete uprights are considered as embedded at their base in the direction of the length of the galleries.

If, in this case, at the top of the upright there is a hinge, then with a change in the temperature the displacement at the top of the upright is equal to the change in the length of the span structure or connected by the hinges of the span structures at the section from the considered upright up to the fixed support.

If, at the top of the upright there is a movable support of the rolling type, then the horizontal load on a upright is determined under the assumption of an arbitrary coefficient of friction of the rolling support, equal to 0.05-0.10.

If, at the top of the upright, there is a mobile metallic support of another type (for example, tangential), then the horizontal load on the upright is determined under the assumption of an arbitrary coefficient of friction equal, in this instance, to 0.5. In this case the displacement and the load which correspond to the change in the length of the span structure (with a change in temperature) are also determined under the assumption of the presence of a hinge at the top of the upright. If the last load is less determinable in the first case (under the assumption of a coefficient of friction equal to 0.5), then in the calculation displacement is assumed, determined under the assumption of the presence of a hinge at the top of the upright.

CHAPTER XI

HOPPERS

1. General Information

The hoppers and receiving funnels of various designs used in the mining industry enterprises, can be distinguished as follows:

receiving funnels for bouldery ore, gained with large clayey inclusions;

receiving funnels for the ore in small particles;

receiving, predominantly skip, hoppers for receiving ore from underground workings;

receiving-loading hoppers for the stems of the mine shafts;

suspension funnels, available in the various joints of the building plan;

intermediate accumulating hoppers;

loading hoppers with the distribution of the mine by motor transport;

loading railroad hoppers;

hoppers for special purposes.

From diagrams and forms of hoppers and receiving funnels one ought to note separately the forms of receiving funnels (Fig. 256a, b, c), receiving skip hoppers (Fig. 256d, e), suspension funnels (Fig. 257a, b, c); one-sided (Fig. 258a) and double-sided hoppers with lateral unloading (Fig. 258b, c); very simple hoppers with a central unloading (Fig. 259a); hoppers with transverse movement of highway transportation (Fig. 259b) and hoppers with central unloading into half-cars at a standard guage (Fig. 260a, b).

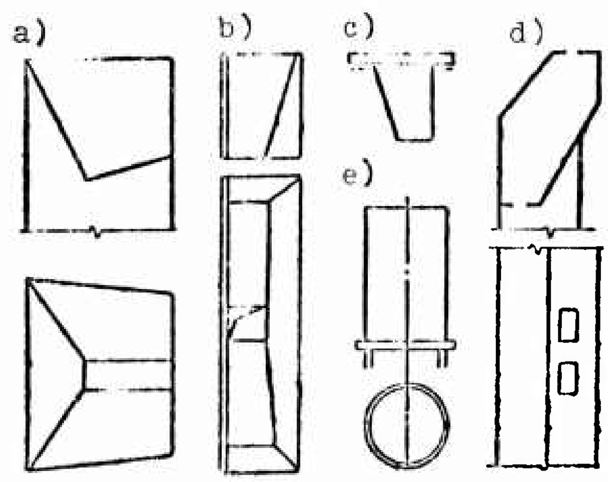

Fig. 256. Forms of hoppers.

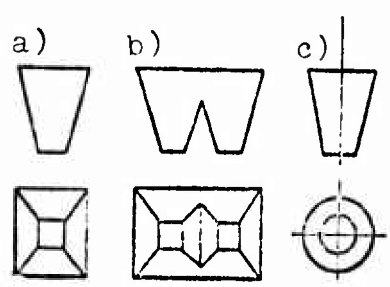

Fig. 257. Forms of hopper-funnels.

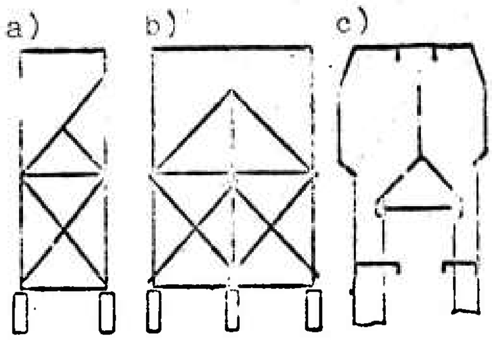

Fig. 258. Forms of hoppers with a lateral unloading.

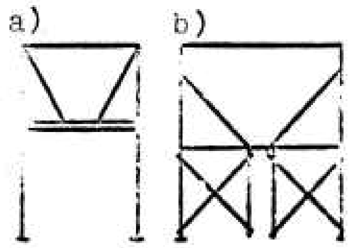

Fig. 259. Forms of hoppers with central unloading.

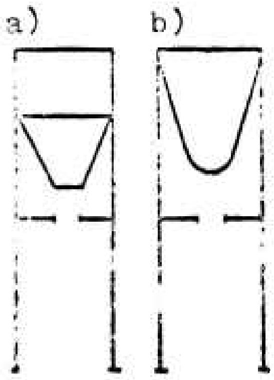

Fig. 260. Forms of hoppers with central unloading in half-cars at a standard guage: a) with pyramidal funnels; b) suspension parabolic hopper.

Wooden, wood-iron-concrete, metallic, reinforced concrete and ferroconcrete hoppers are used. The latter can be erected using sectional, sectional-monolithic and monolithic designs.

As is known, hoppers are considered as storage bins of loose material with the height of the bin within the limits of one and one-half of the measurement of the construction cell by design. Relatively small capacities with an increased ratio of the height of the cell to its width at a height up to 8-10 m are also considered hoppers.

2. Receiving Funnels

It is possible to use several receiving skip ore hoppers as an example. The receiving skip storage-type ferroconcrete hopper, presented in Figs. 261 and 262, has rectangular outlines with sizes by design 6.5 × 10 m. In a transverse cut (Fig. 261) the capacitive part of the hopper has a polygonal form. The receiving funnel is set near the pile driver machine. The longitudinal cut of the hopper is represented in Fig. 262. The foundations of the hopper are spaced at 18 m, which is determined by the requirements of their spread with the arrangement of the outside overall size of the shoring of the mouth of a shaft stem and other equipment of the stem of the mine shaft. In connection with the large loads of the capacitive part of the hopper and the considerable intrinsic weight of the ferroconcrete hopper, the size of each foundation by design constitutes 6 × 11 m.

With soils of increased bearing capacity the hoppers have vertical uprights and their forms are simplified (Fig. 263).

Hoppers of the described type — heated; the underhopper area, located at a height of 6.5-7 m over the planning level, is heated. Here, locks, which are pneumatic on one side, and which on the other, have a spare hand drive, are installed. Loading is done in half-cars of standard guage track, located directly under the hopper. At heights of 3.5-4 m above the planning level is where the control platform is located. The operator located here governs the locks and from his post directly observes the process of loading.

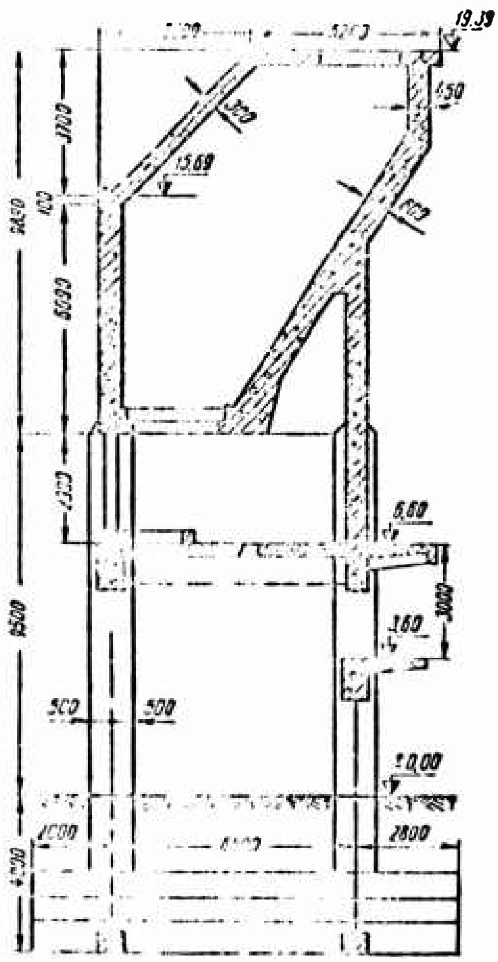

Fig. 261. Receiving skip storage-type ferroconcrete hopper; a cross-section.

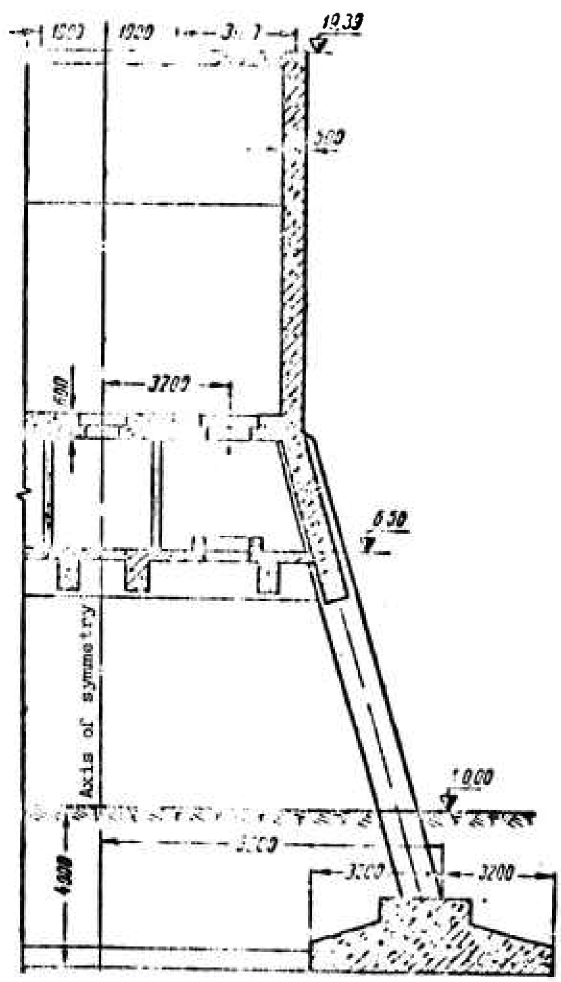

Fig. 262. Receiving skip storage-type hopper; longitudinal section.

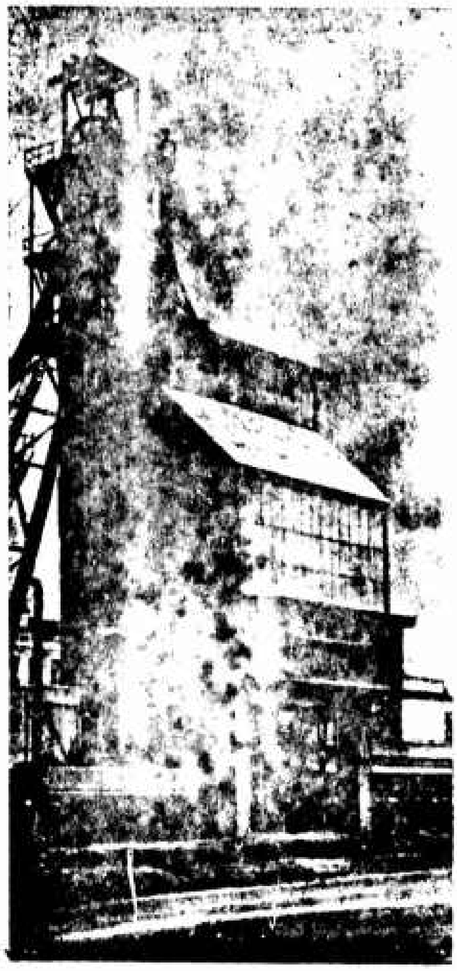

Fig. 263. Receiving skip ferroconcrete hopper with soils of increased bearing capacity.

The soils of the foundations of the described hoppers are characterized by a calculated strength within the limits of 2-4 kg/cm^2. Frequently, the soils are not as strong. Thus, one receiving skip hopper of the same series, presented in Fig. 264, is located in the immediate vicinity of the receiving funnel of crushing mill. The depth of the funnel location is 18 m lower than of the surface level, and the revetment to the skip hopper is 14 m. In connection with this the hopper is based on moveable ferroconcrete caissons, sunk to 13 m lower than the level of planning. The distance between the caissons constitutes, lengthwise to the construction, 10 m, which corresponds to the step of the uprights of the hopper. The uprights of the skip hopper in this case are vertical and are arranged in a grid 6.5 × 10 m.

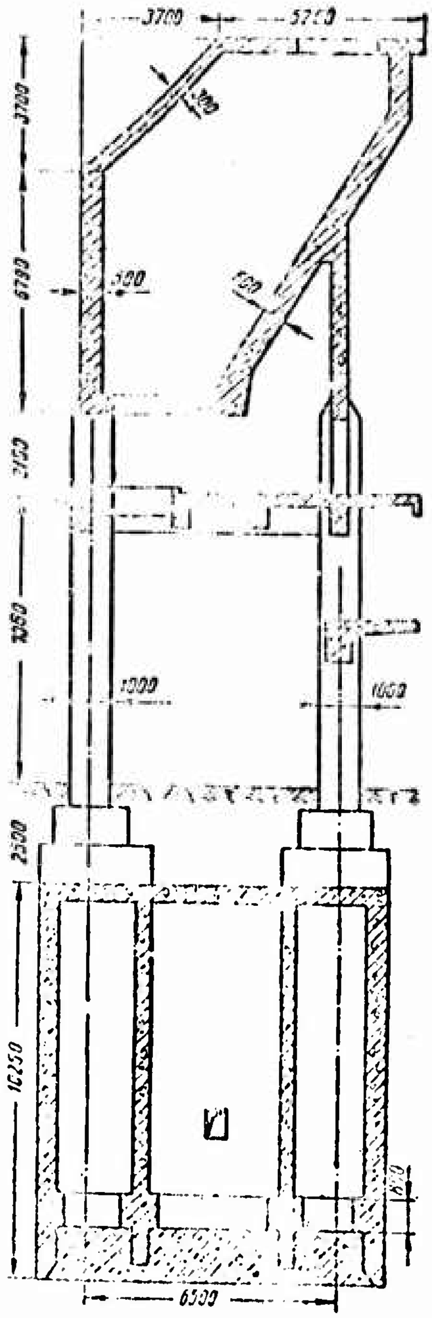

Fig. 264. Receiving skip ferroconcrete hopper under conditions with soils having a low bearing capacity.

The ore, discharged from the overturning skips, falls into the receiving trough supported by a metallic frame. The inclined bottom of the trough is lined with steel sheets, placed on a shock-proof layer, which was made from a layer of wooden uprights, 240 mm thick. Shock-proofing is attained because of the deformations of the transverse bending and the transverse compression of the wood. Actually, the hopper also has a lining made from sheet steel, placed

on a continuous layer of wooden uprights 180 mm thick (Fig. 265a, b). Rolled double-T No. 18 beams are placed on the thicker layer of wood, strengthened along the bottom and walls of the hopper with the help of matching parts. The parts of the lining are presented in Fig. 265c, d. As it appears from the given diagrams, the junctions of the sheets lining by design are located on the double-T beams, along the axes of which the sheets are welded with inclined or vertical seams on the flanges of the double-T beams. The junctions of the sheets of lining are overlapped by continuous vertical cover plates made from band metal having a section, 10 × 100 mm, arranged parallel to the slope.

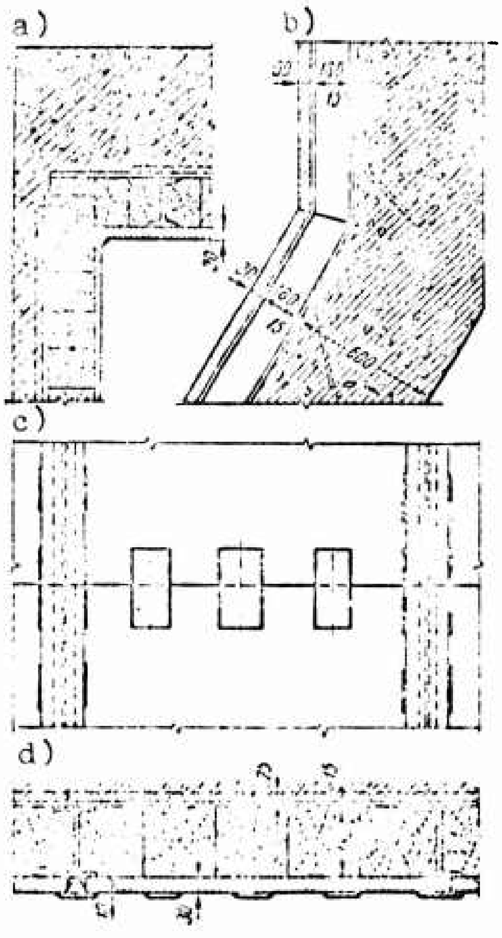

Fig. 265. The lining of a receiving hopper.

In a mining industry cylindrical skip receiving hoppers are used. Sometimes, near the stem of the mine shaft two cylindrical hoppers are set on a ferroconcrete pedestal (Fig. 266). A lightweight wall barrier is set at a distance of 0.8 m from the metallic cylinder. The space between the hopper and the enclosing wall is heated. More contemporary version of the described type are the ferroconcrete cylindrical hoppers, also arranged in pairs in the stem of the mine shaft (Fig. 267). The diameter of the cylinder in this case is equal to 8 m, a height of 10 m, a thickness of the walls of 0.3 m. Heat insulation is fastened directly to the ferroconcrete cylinders. The height of the cylindrical hoppers attains 40 m with a diameter of 12 m.

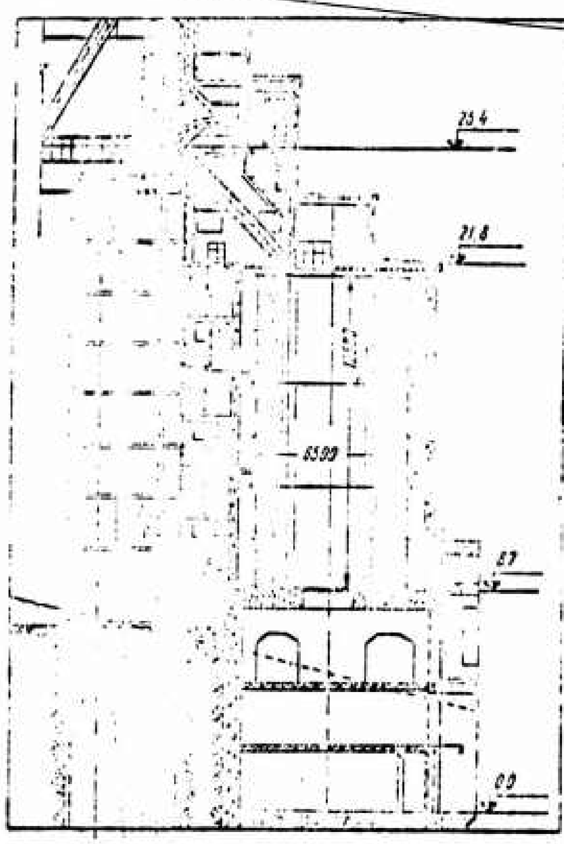

Fig. 266. Cylindrical skip receiving hoppers.

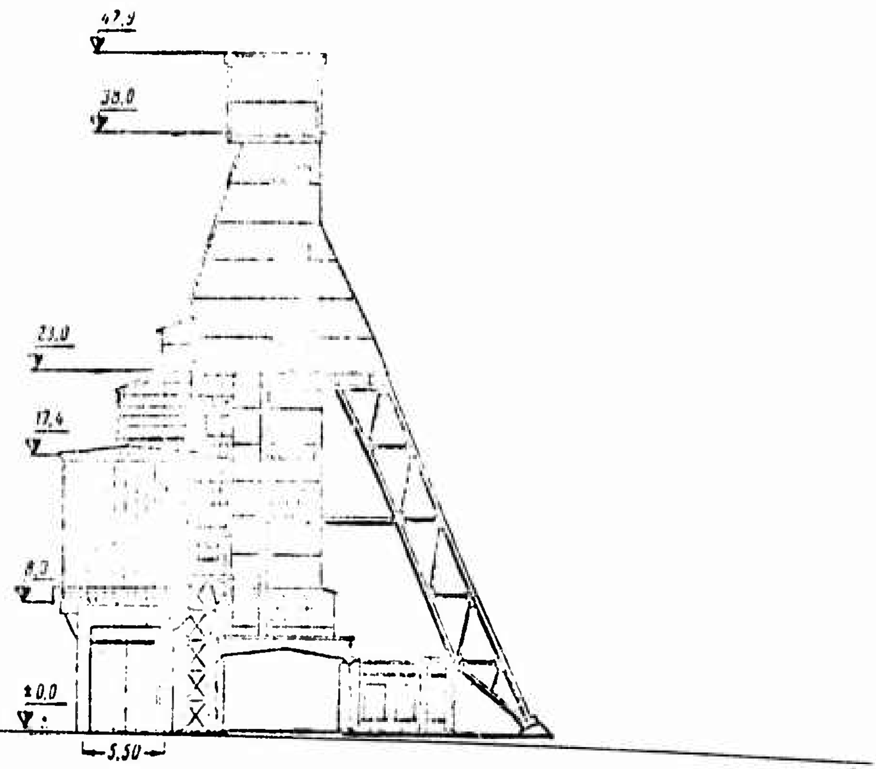

Fig. 267. Cylindrical ferro-concrete skip hoppers.

The funnels, which serve for receiving the ore from the open pit, are situated in the most unfavorable condition from the point of view of the impact pressure of the falling ore lumps. The exception is the funnels of the individual sorting devices, utilized for receiving ore of small fragment sizes. With the transport of such ore by dump trucks having a load-carrying capacity up to 5 t, the receiving funnels in a number the series of cases do not have amortization. The general diagram of such receiving equipment is represented in Fig. 268, and the details of the funnel are given in Fig. 269. The funnel is set on a ferroconcrete flooring and represents a frame made from No. 24 channel uprights which form the horizontal straps of the funnel. The rails are located on channel uprights at a distance up to 0.5 m and on the uprights the steel sheets of the planes of the funnel are placed; the thickness of sheets is 20 mm. The horizontal supporting frame is formed by the channel uprights and by No. 30 double-T beams. The latter together

with welded-on rails form the control lattice of the funnel. In the plane of the rails transverse rail bracer are spaced every 0.5 m. The supporting frame is fastened to ferroconcrete flooring by anchors. The weight of the funnel and control lattice constitutes about 11 t; the sizes of a funnel in a plane is 3.0 × 4.3 m; the height of the funnel is about 3 m.

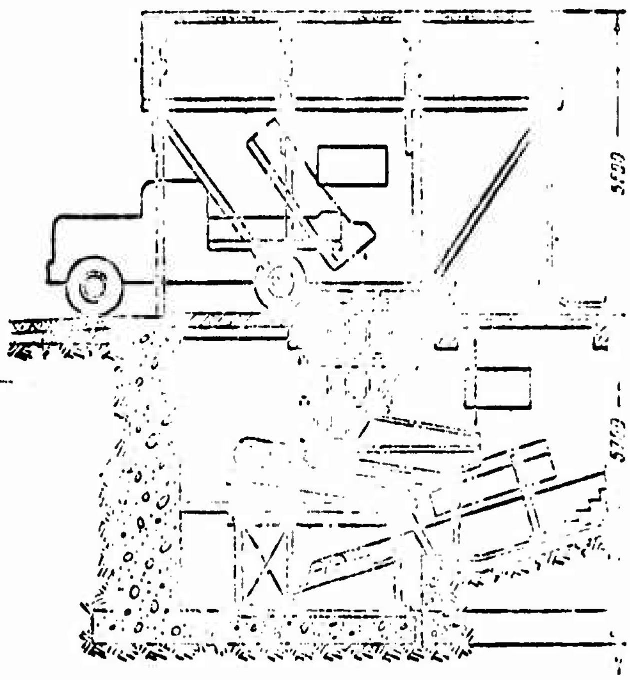

Fig. 268. The funnel of a sorting device for receiving ore of small particle size.

The simplest funnel, receiving ore from the half-car, is represented in Fig. 270. The funnel is inscribed in the support wall of the crushing device. Three planes of the funnel are found on this concrete support wall. The fourth (vertical) plane is formed by the metallic frame, anchored in the support wall. A trough is also fastened to this wall from the receiving funnel to the crusher. The support wall shown in the figure was actually used under the described conditions continuously for about 30 years, and the wall abuts a rocky cliff the larger part of its height. This can explain the comparatively small sections of the support wall.

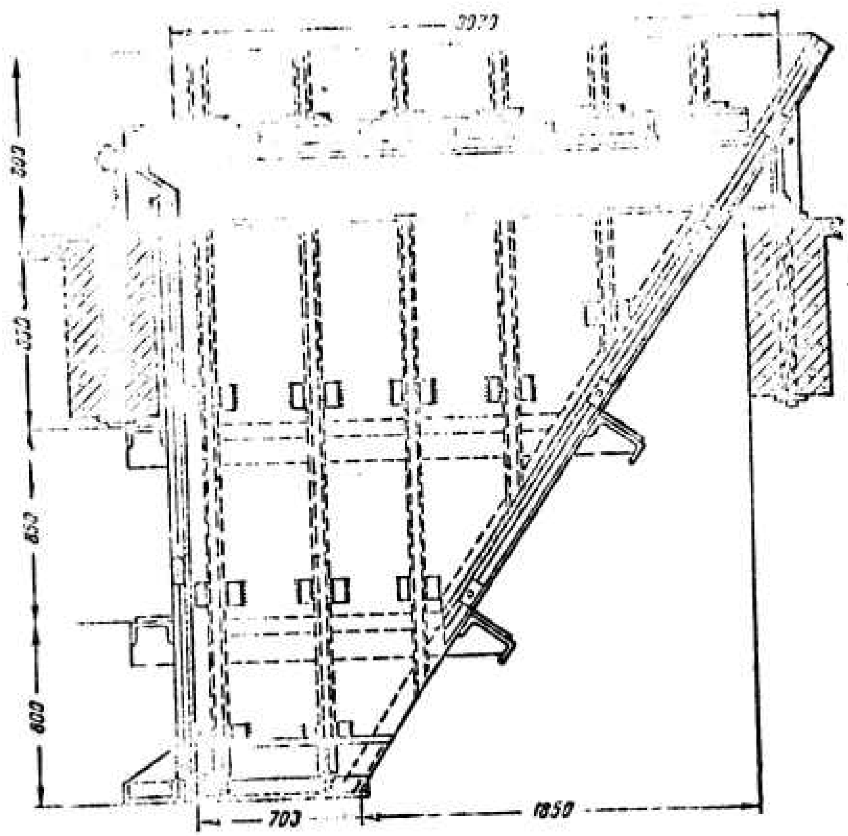

Fig. 269. Details of the receiving funnel for ore of small fragment size.

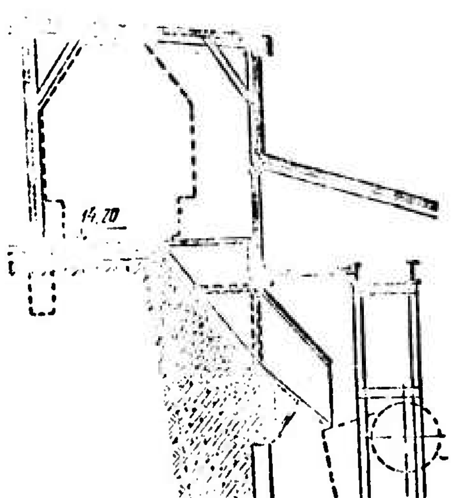

Fig. 270. The simplest funnel, combined with a relieving wall, for receiving the ore from the half-cars of standard guage.

The funnel, which receives the ore from the quarry, is represented in Fig. 271. The lamellar feeder installed under the funnel supplies the ore to a crushing device. The latter is composed of two crushers, arranged according to the height in the diagram. The foundations of the frame of the receiving funnel is located at the level of the bases of the foundations of the crusher.

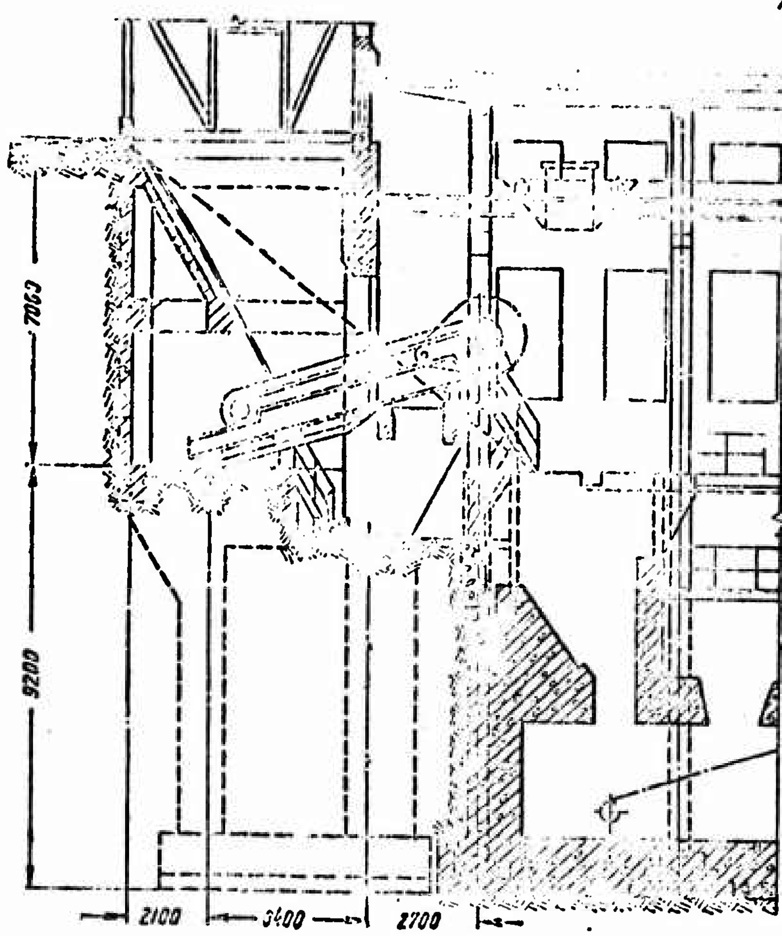

Fig. 271. A funnel for receiving ore from a quarry.

Figure 271 also characterizes the joints of the feeder device and the tying in of the latter with the receiving funnel. One should turn his attention to the arrangement of the beams of the landing under the feeder. The position of beams is tied to the joints of the feeder and it assures the dismantling of the parts of the equipment.

The more complex units of the device are shown in Fig. 272. The receiving funnel, analogous to that described above, is located between the crushing device and the stem of the mine shaft. The receiving of the ore is done from the mine shaft through the skip funnel 1 from a quarry through aperture 2. The foundations of the receiving funnel 3, crusher 4 and the shoring of the mouth of the stem of the mine shaft 5 are located at one level. The lining and shock absorption of the receiving funnel are analogous to that described above, with the distinction that the bracing beams of the lining 6 are set beyong the funnel in a direction towards the feeder and continuous along its entire extent.

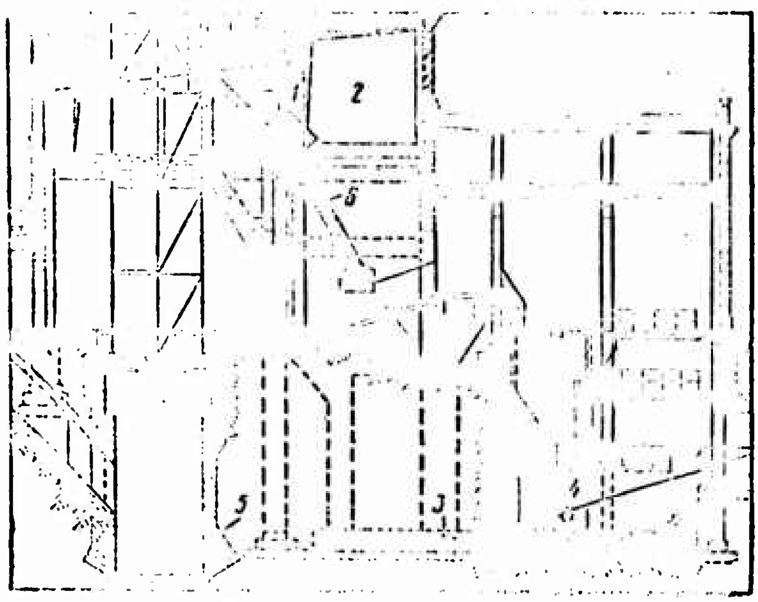

Fig. 272. The receiving funnel, located between the crushing device and the stem of the mine shaft.

Figure 273 presents a large receiving funnel, receiving ore from a quarry and mine shaft. A half-car 1 is located on the approach to the funnel and over the funnel. The height of the fall of the ore in this case is equal to about 10 m. The adjacent haulage gallery 2 serves to supply the ore by electric locomotives and is made up of trolleys from the mine shaft. In the chutes (not shown

in Fig. 273) the ore from the mine shaft also will pass into the receiving funnel 3. The depth of a funnel is 17.1 m. The crushing housing and its foundations adjoining to the funnel are set at the same level.

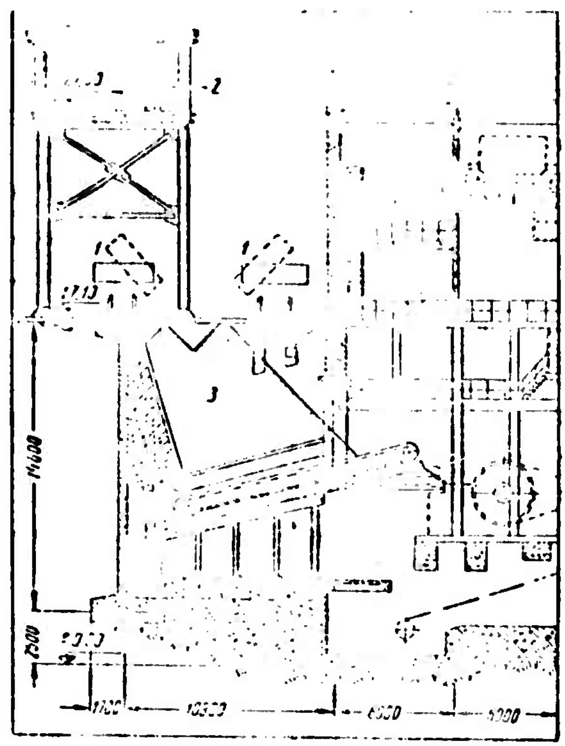

Fig. 273. The receiving funnel for receiving the ore from the quarry and the mine shaft.

3. Loading Hoppers

Steel loading hoppers with central loading, relative to the installation weight, are easy to mount, and the units of any loads and shapes can always be installed. The utilization of steel, however, is expedient only for the capacitive part of the capital hoppers, in connection with the fact that recently there have appeared complex designs of hoppers, whose frames of simple form are made from reinforced concrete, and capacitive part, made from metal.

A very simple metallic ore hopper with a central loading is given in Fig. 274. Several such noppers are installed for receiving ore direct from the gallery or deckhouse building according to the diagram, approximately corresponding to that in Fig. 195. Under these conditions the basic task is the possible decrease in the height of the hopper. The height in such cases, unlike that for normal loading hoppers for receiving ore by conveyors, usually does not exceed 10-12 m. The spacing of the uprights in a longitudinal direction constitutes 3-4 m, sometimes 6 m. In the latter case the introduction of longitudinal beams and intermediate trusses is necessary, which substantially complicates construction. The spacing of the uprights less than 3 m is disadvantageous. In such hoppers a device is necessary consisting of special mechanized locks, which operate somewhat according to the modified principle of Chinese chests. Without these locks the back end of the hopper and operating landing are designed according to the description in Figs. 287, 288. Despite its primitiveness, such a scheme of loading is one of few feasible over at low temperatures and with a moist ore. One should pay attention to the fact that the operation of the described structure differs using specific loads. In the winter period the impact perssure is possible and is heavier than usual when the pieces of ore congeal; during the unloading of the vessels one should not exclude the possibility of using small blasts. Under such conditions the utilization of only steel designs in the capacitive part is possible. Under specially severe conditions, with a high content of moisture in the ore, the hoppers should be warmed. In the latter case the scheme of the structure usually differs with warm underhopper equipment for locks or feeders and also by the increased height of the hopper, equal to 12 and predominantly 15-18 m.

With the increase in the height of the hopper, the capacity of the structure substantially increases, the expediency arises for the use of normal loading hoppers, the capacitive part of which is represented usually by a flexible parabolic cover, suspended along the extreme upper position of the formed cover lengthwise to the beams of the hopper. The cover can be continuous and made from sheet

metal. However, the presence of the lining and shock absorbers usually leads to the need to make the hopper from individual flexible supporting strands of band metal, arranged evenly lengthwise to the structure.

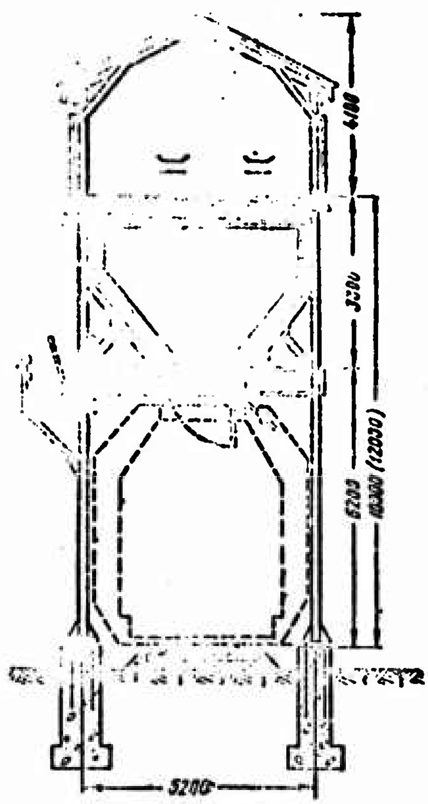

Fig. 274. A very simple metallic ore hopper with central loading.

The outline of the curve of a transverse section of the flexible cover or separate flexible strand should be selected so that the latter with the filling of the hopper would operate only under tension. The more accurate the form of the strand is selected, the less the stresses, and the change in its outline during operation.

The presentation on the metallic loading hoppers with a flexible capacitive part is given in Fig. 275, where a parabolic hopper of a coal mine shaft made of three cells, 6.3 × 7.0 × 15.5 m, with an overall capacity 600 t, is presented. The metallic transverse frames of this hopper sustain the loads of the capacitive part through the longitudinal beams with a height of 1970 mm. In connection with the

attachment of the sheets to the bottom to the lower strap of the beam, the latter is reinforced with vertical sheets, and thus represents a horizontal double-T beam, capable of sustaining horizontal components of loads from the capacitive part. A similar abutment of a flexible bottom to the longitudinal beam is necessary in connection with the rectangular volume of storage over the flexible part.

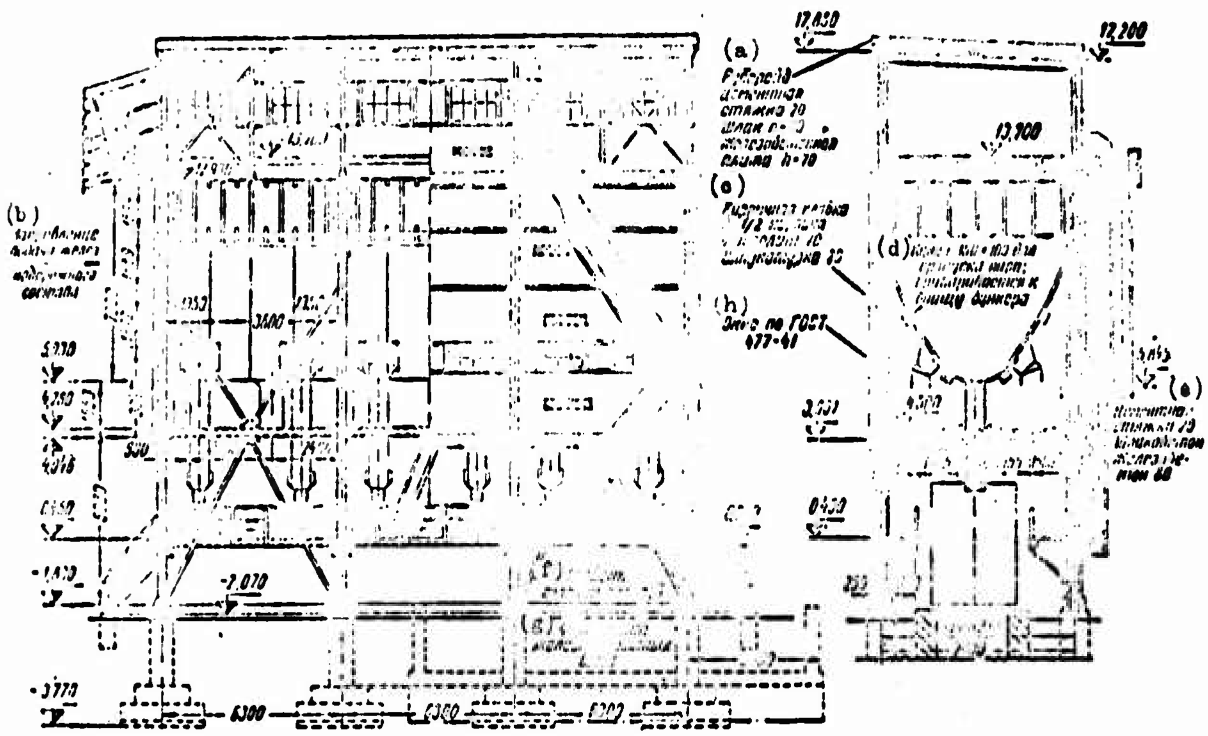

Fig. 275. A metallic loading hopper with a flexible capacitive part having a capacity of 600 t of coal.

KEY: (a) Rubberoid cemented coupler 20, cinder p = 80, ferroconcrete slab n = 70; (b) Feed control of the rolling stock; (c) Brick facing; 1/2 brick; stucco 20; (d) Duct 150-100 for the passage of steam; it is welded to the bottom of the hopper, (e) Cemented coupler 20, slag-concrete 80; (f) Level of the head of the rail; (g) Foundation of the railroad beds; (h) According to GOST 477-41.

The ratio of the height of the parabolic part of the hopper to its width in the given example constitutes about 0.7. The expenditure of metal per 1 m^3 capacity of hoppers constitutes about 0.18 t; per cell with a capacity of 200 t – 0.25 t, and per five cells with a sum total capacity of 1000 t – 0.16 t. On the average, the expenditure of metal to loading coal hoppers with a flexible capacitive part,

constitutes about 0.2 t/m^3. With the volumetric weight of coal at about 0.9 t/m^3, the average expenditure of metal constitutes approximately 0.22 t per 1 t of capacity.

It is possible to show that the indexes of the expenditure of metal structures decrease with an increase in the overall sizes and capacity of the hoppers. The largest ore hopper with a parabolic capacitive part is characterized by a width of 18 m, by a height from the bottom of the capacitive part up to the top of the hopper plate of 13.9 m, by a height of the parabola of 12.53 m, the length of the hopper of 552 m. The capacitive part of the hopper is suspended to an inclined metallic beam with a span of 12 m at a height of the beam of 2.9 m. The wall of the beam is represented by a sheet, 2860 × 30 mm, the upper strap – by a sheet, 800 × × 40 mm, the lower strap is shifted according to the conditions of attachment of the beam to the upright from the lower edge of the wall by 900 mm; the strap has an overall width of 400 mm and is composed of two sheets having a thickness of 40 mm. The continuous sheets of the capacitive part have a thickness of 16 mm, in the lower part of the hopper in the sections of the apertures this thickness is doubled and is equal to 32 mm.

A normative load from the weight of the ore in a hopper constitutes about 200 thousand t. The weight of the metal structures of the capacitive part constitutes, in this case, about 6000 t; the overall weight of the metal structures of the hopper span – about 23,000 t. The corresponding indexes of the expenditure of metal structures per ton of ore is approximately 2 times lower by comparison with the indexes of the expenditure of the metal structures of the coal hoppers, given in Fig. 275.

Ferroconcrete loading hoppers. Figure 276 shows a ferroconcrete coal loading hopper. Depending on the productivity of the enterprise, these hoppers have 3 cells of an overall capacity of 540 t, 4 cells of an overall capacity of 720 t and 6 cells of an overall capacity of 1080 t. In order to get the indexes of the expenditure of the

chief material, specifically monolithic and precast reinforced concrete per 1 t of capacity of the hopper, in this case, one ought to determine these indexes from the increases in the expenditure of reinforced concrete and capacity of the hopper.

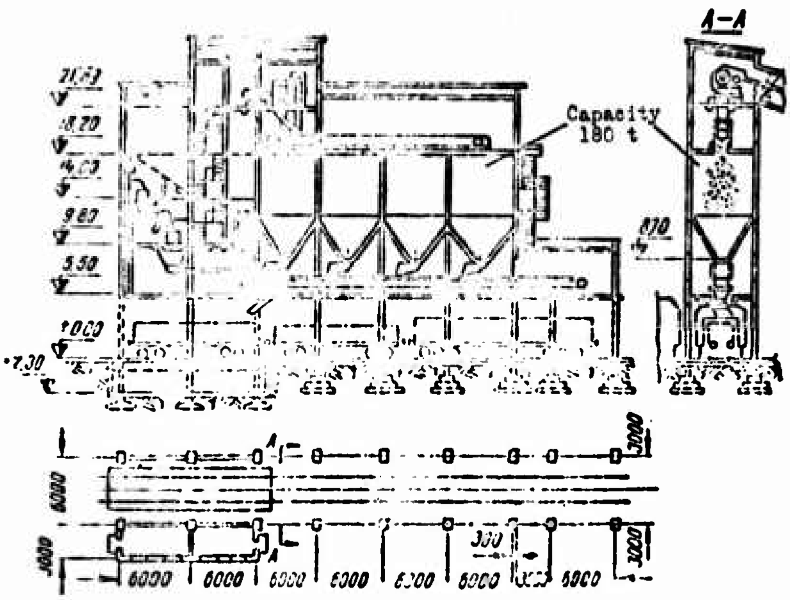

Fig. 276. A ferroconcrete loading hopper with a capacity of 720 t along with central loading.

For example, with an expenditure of a monolithic and precast reinforced concrete for a hopper of type KB-18 (capacity of 1080 t) 1026 m^3, and for a hopper of type KV-17 (capacity of 720 t) 823 m^3, for an increase in the expenditure of reinforced concrete, equal to 203 m^3, the increase corresponds to a capacity of a hopper, equal to 360 t. The index of expenditure of reinforced concrete per 1 t of capacity of the examined coal hopper is equal to 0.6 m^3/t; however, allowing for the need of a stair cage device of small overload (taken into consideration in all the following examples) one ought to consider that the expenditure of reinforced concrete in this case will constitute about 0.7 m^3 per 1 t of capacity of the hopper.

In this way the same indexes of the expenditure of reinforced concrete also for other intervals of capacity can be determined.

Figure 277 shows a ferroconcrete monolithic ore hopper. Hoppers of this type are widely used in Krivoy Rog mines. The height of the hoppers over the level of the head of the rails of the loading passages is usually equal to 12.5-14 m, the bottom of the discharge apertures – at heights of 5.0-5.5 m, the height of the capacitive part, 7.5-8.5 m, the width of each of the two sections, 4-5 m.

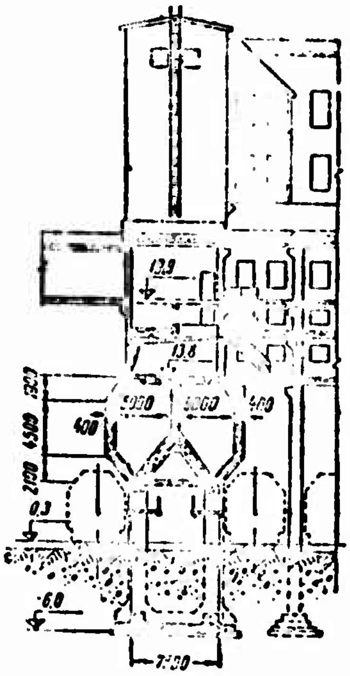

Fig. 277. Monolithic ferroconcrete ore loading hopper with two-sided loading.

This type of hopper has transverse continuous walls, the lower edge of which in a number of cases is located somewhat lower than the discharge opening, and the top coincides with the upper plate of the hopper, which is the floor of the hopper gallery. The thickness of the walls of the hopper predominantly constitutes 0.3-0.35 m. The walls and bottoms are lined with metallic sheets. The thickness of the sheets in the sections of the bottom of the hopper is 12 mm, on the walls of the hopper, 8 mm. The loading ore hoppers described

here are characterized by comparatively high input labor, associated with the need to complete the ferroconcrete work of considerable volume. The expenditure of the reinforced concrete, needed to erect hoppers of this type, is equal to 0.55-0.7 m^3 per 1 t capacity, and frequently about 0.6 m^3/t. The expenditure of sheet and other metal, necessary for lining the bottom and certain incomplete lining of walls of the hopper (and also for the manufacture of stairs), constitutes 0.06-0.08 t per 1 t capacity, of which about 0.02 t is for the lining of only the transverse walls.

The utilization of precast reinforced concrete in the described ore hoppers is associated with a number of difficulties caused by the relative complex forms of the hopper. Figures 278 and 279 illustrate the precast ferroconcrete ore hopper, the forms of which are somewhat simplified in comparison with the above described because of the change in the outlines of the top of the capacitive part. Simultaneously, because of the increase in the grade of concrete and content of steel framework the thickness of the plates of the capacitive part is reduced to 260 mm. However, even in this instance, the precast designs of the construction are characterized by relative complexity (Fig. 279), and the expenditure of reinforced concrete per 1 t of capacity of the hopper, somewhat reduced against the given indexes, is still very high.

Figure 280 shows a precast ferroconcrete loading hoppers. There are three spans over 7.5 m across the hopper; the overall width of the structure is 22.5 m, the length is 48 m.

The distance between the uprights in a longitudinal direction is equal to 6 m. Three loading railroad tracks are located on the spans of the construction. The height of the construction at one half of its length constitutes 23.8 m, and the other – 28.2 m. The walls of the housing and its enclosed volume are located at heights of 6 m and more over the head of the rails of the loading tracks. The construction volume of the enclosed part of the construction is 34.0 thousand m^3; the overall volume of the construction is 40.9 thousand m^3.

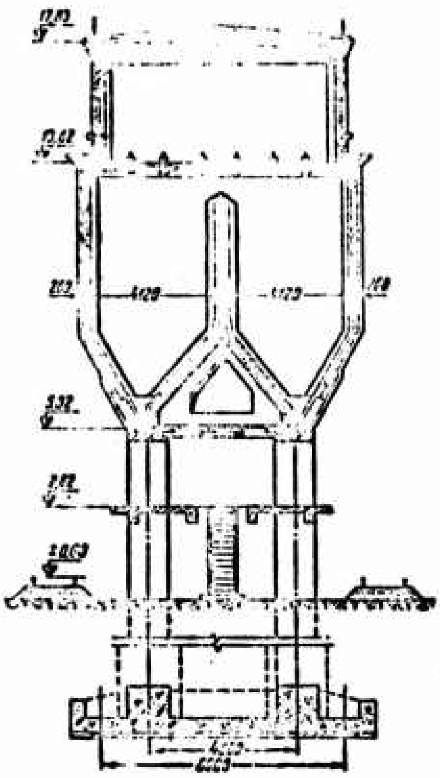

Fig. 278. Precast ferroconcrete ore loading hopper with two-sided loading.

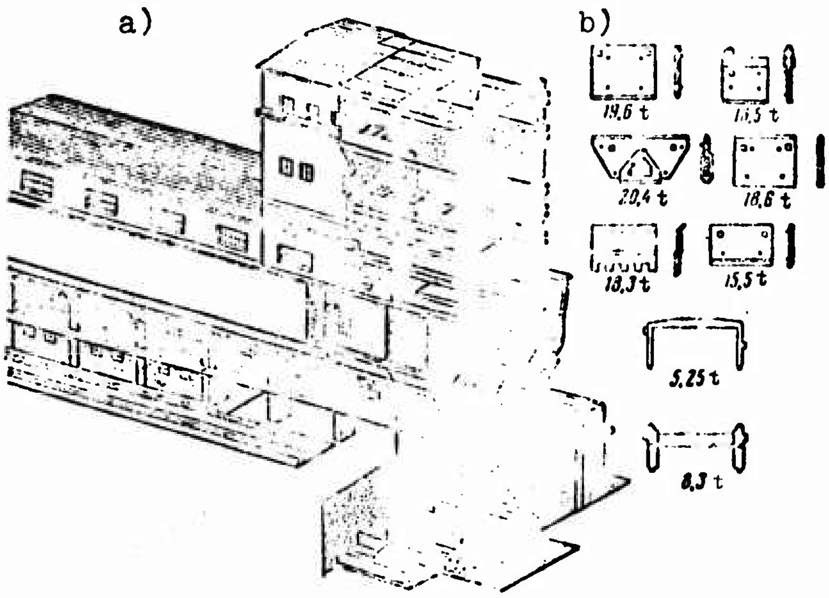

Fig. 279. Precast ferroconcrete ore loading hopper: a) the axonometry of the designs of a precast hopper; b) the precast parts of a hopper.

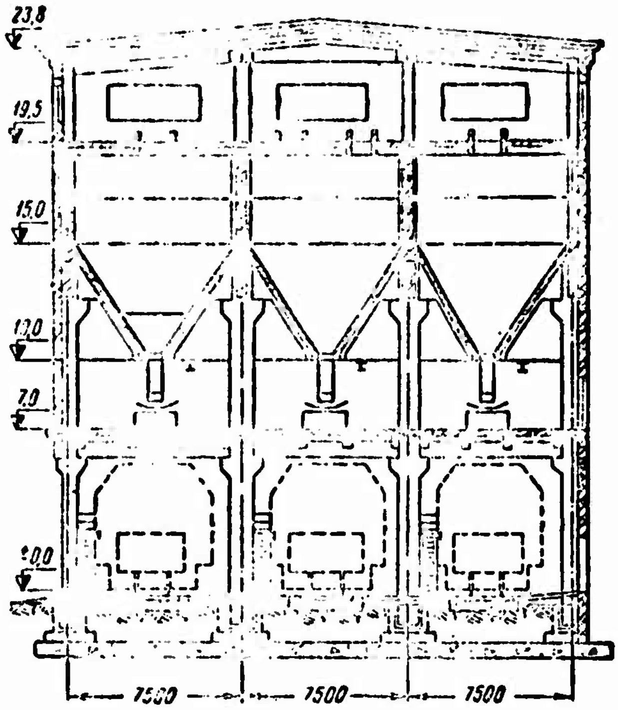

Fig. 280. Precast ferroconcrete triple-span loading hoppers.

The foundations of the construction are presented as flat ferroconcrete plates having a thickness of 0.7 m, set at a depth of 2.6 m from the head of the rails. Double-reduction sockets are set on the plate, the upper steps of which are brought to the level of planning and are provided with sleeves for the enclosing the precast ferroconcrete double-level columns.

In the plane of the main frames the flooring crosspieces rest on the cantilevers of the columns at a level of 7 m, and the transverse cross pieces of the second course, the top of which is at a height of 15 m above the head of the rails, also rest on them. Also, the longitudinal beams, the top of which is found at the 14.35 m mark, rest on the console of columns.

The basic longitudinally inclined plates of the bottom of the capacitive part lie on longitudinal beams and are attached to the crosspieces of the second tier (Fig. 281). Within the limits of each six-meter section of the capacitive part, lengthwise to the

construction, there are two discharge openings, limited by three transverse ferroconcrete dissectors, which lie on the longitudinal plates of the bottom. Above these plates of the bottom and over the crosspieces of the second tier, flat vertical ferroconcrete slabs, 250 mm thick, are set, which separate the capacitive part from the cells. In the whole housing there are 18 complete cells, similar to those shown in Fig. 280; furthermore, four cells have only a prismatic part.

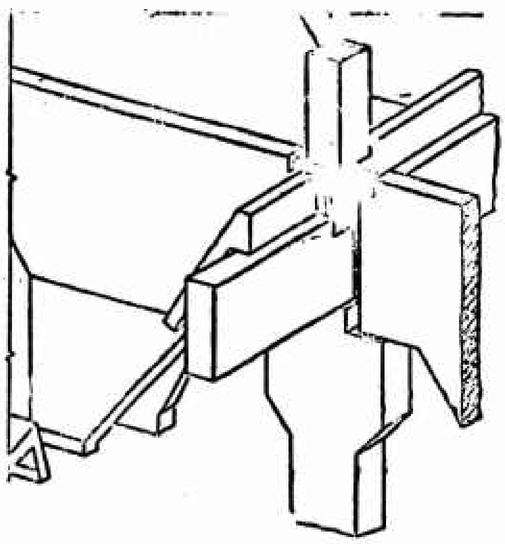

Fig. 281. A unit of the designs of precast triple-span hoppers.

The described precast ferroconcrete hoppers are characterized by the following indexes, pertaining to their 1 t capacity: the expenditure of precast reinforced concrete (with framework in the amount of 0.17 t/m^3) constitutes 0.28 m^3, the monolithic reinforced concrete (with steel framework in the amount of 0.1 t/m^3) 0.22 m^3, whole reinforced concrete 0.5 m^3; the expenditure of the lining and the steel designs, 0.04 t, the expenditure of the metal in the steel framework (predominantly St. 5 steel) and matching parts, 0.078 t, the overall expenditure of metal, 0.12 t.

The weight of the ferroconcrete and steel designs of the construction is equal to 1.34 t, and allowing for the weight of the brickwork, 1.64 t per 1 t of capacity.

The quantity of the type and dimensions of precast ferroconcrete designs are equal to 58, the average number of elements of one type and dimension - 16. The heaviest precast designs with a weight within the limits of 4.8-21.7 t are characterized by 39 types and dimensions with an average number of elements of one type and dimension, equal to 8.

Figures 282 and 283 show an actually used precast ferroconcrete loading hopper with a flexible parabolic capacitive part. The height of the parabola is 6.37 m; the width is 5.9 m. The funicular curve of the suspended hopper is determined by the equation of the parabola, $x^2 = 1.366y$. The height of the hopper from the level of the floor of the hopper transport gallery to the head of the rails of the loading track of normal gauge is 15 m. The length of the expansion block of the hopper is 42 m. The block has connections in the longitudinal direction. The hopper is warm with heating for all of the structures located above the loading tracks.

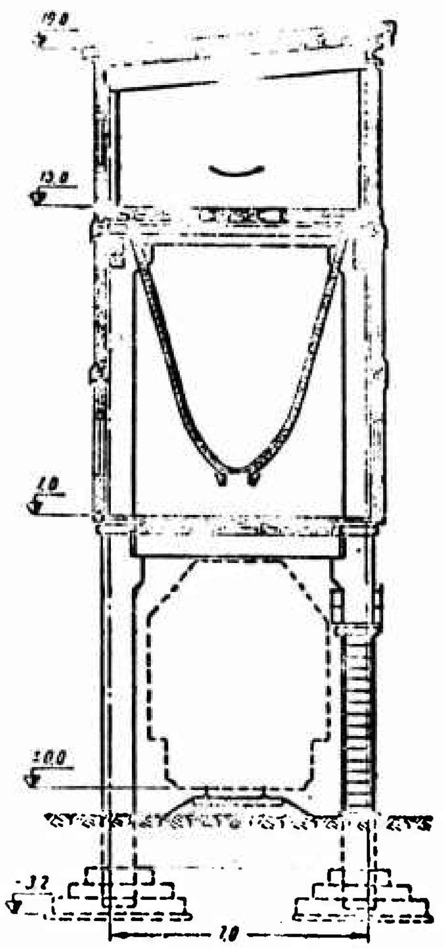

Fig. 282. Precast ferroconcrete loading hopper with a suspension parabolic capacitive part.

Fig. 283. View of a ferro-concrete loading hopper with a suspensed parabolic capacitive part.

The coupling of parts in the units of the structures of a hopper is done with the help of metallic cover plates and matching parts, weldable during installation of the frames of the hopper. The columns of a precast hopper have one junction plate the weight of the columns and the longitudinal beams is equal to about 15 t. A device of the capacitive part, which is characterized by the simplest support lengthwise to the beams, and by the absence of torsion of the latter (Fig. 284).

The barrier of the walls, flooring, planking – precast. For the filling of the walls precast shields, composed of two layers of asbestos-cement of corrugated sheets of a reinforced profile are used. For the warming of the walls and flooring slag mats are used.

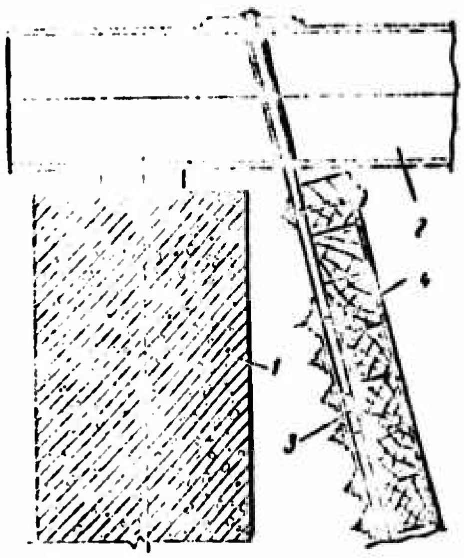

Fig. 284. A part of the support of designs of the capacitive part on longitudinal ferroconcrete beams of the hopper: 1 – main beam; 2 – double channel uprights, No. 30a, set at a spacing of 1750 mm; 3 – suspension of 220 × 18 mm; 4 – lining.

During building, there is a second hopper with a parabolic capacitive part, such as that described above (Fig. 283). The length of the capacitive part of the second hopper (divided into 4 expansion blocks) is equal to 120 m, its capacity is equal to 5000 t. Both hoppers are characterized by the following main indexes, based on a 1 t capacity: the expenditure of the structural and monolithic reinforced concrete is 0.22 m^3, the weight of the steel designs and linings, without considering the weight of the metallic framework of a wall guard (about 0.01 t) is 0.06 t; the overall investment of steel, allowing for the steel framework is 0.10 t; the weight of the ferroconcrete and steel designs and lining is 0.63 t, the weight of the support designs and of wall guards – about 0.7 t; the number of type and dimensions of the precast ferroconcrete designs (without considering the flooring slabs) 15, the average number of elements of one type and dimension (also without the flooring slabs) is 28.

In this and other hoppers monotonous designs of the capacitive part based on the type presented in Fig. 284 are used. The basic part of this design represents a parabola made from band metal with a beam-spreader (metallic or ferroconcrete), which rests on the longitudinal ferroconcrete beam of the hopper. The following varities of the design are possible:

1) lining on a wooden slab, set in metallic suspensions with a parabolic outline, spaced at a distance predominantly of 1.5-2.0 m and up to 3 m;

2) reinforced lining, set directly on metallic suspensions of a parabolic outline;

3) lining on a flooring made from prestressed reinforced concrete, resting on metallic suspensions of a parabolic outline.

The latter design of the capacitive part of the hopper can be used with the distances between the axes of the suspensions at 3-6 m. The concentration of the metal in this instance with a comparatively small number of parabolic metallic suspensions leads to the possibility of the wide utilization for their manufacture in low-alley construction, structural and high-strength steel in the form, for example, of steel cables with spacing sheets or washers made from steel of brand St. 3, set between the cables and the flooring. Specifically, the following design of a capacitive part is possible: two paired parabolic light suspensions made from low-alloy steel with metallic (or ferroconcrete) spacers resting directly on longitudinal cantilevers of the ferroconcrete cables of the hopper, symmetrically relative to the axis of the tier. The parabolic metallic suspensions support the precast ferroconcrete flooring elements - lining, whose calculated diagram is a double-cantilever beam with a span of 4.8-5.0 m and with symmetric cantilevers, whose extension constitute 0.6-0.5 m. It is possible to show that with this design of a capcitive part, the need completely diminishes for the longitudinal beams of the hopper.

On the sections near the discharge openings continuous sheets are usually installed. The two parabolic steel suspensions, which intersect this section, are made in the following manner:

the parabolic suspension strands made from high-strength steel, and sometimes even in the case of utilizing structural steel, can be accepted as a continuous, sheet made from St. 3 steel maintaining a continuous bend;

parabolic suspension strands made from St. 3 steel, and also from low-alloy structural steel are broken off and jointed with sheets with the help of sheathing using bolts or welds (Fig. 285).

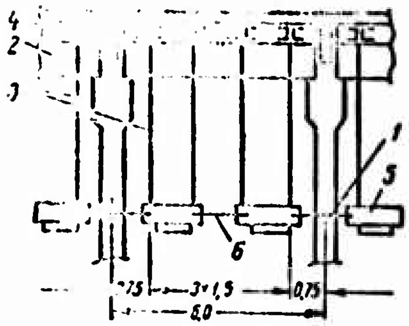

Fig. 285. The diagram of the parabolic capacitive part of the hopper: 1 - ferroconcrete column; 2 – longitudinal ferroconcrete beam; 3 – parabolic curve of the suspension made from band steel, broken at the part 5; 4 – beam-spacer; 5 - continuous sheet at the discharge aperture; 6 – connection between the parts 5.

In all cases the top of the bent continuous sheets should have flanged corners. Usually installed somewhat below the level of the upper board of sheets are longitudinal connections 6, set at a constant distance between the discharge openings (Fig. 285).

Figure 286 illustrates a loading ore hopper with a parabolic capacitive part and with precast ferroconcrete transverse frames of H-shape. The sizes of the structure by design are 6 × 45 m. The hopper itself is laid out over a length of 42 m and consists of seven cells each the size, 6 × 6 m, by design. The capacity of the hopper – about 2500 t. The capacitive part of the structure is formed by the main longitudinal metallic beams of the hopper, set on uprights of the ferroconcrete frames. The longitudinal beams support the metallic hopper of parabolic form and transverse beam-spreaders, to which the structures of the guard of the upper part of capacitance body are fastened. During the usual filling of the hoppers, the transverse beam-spreaders are in the ore over the largest part of their length.

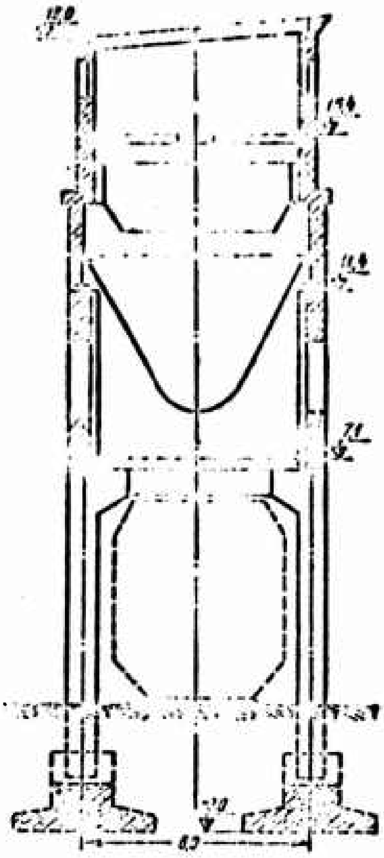

Fig. 286. Precast ferro-concrete loading ore hopper with a parabolic capacitive part and with H-shaped frames.

The foundations of the loading hopper – a ferroconcrete footing, lain at a depth of about 3 m from the level of the head of the rails of a loading railroad track. The filling of the walls – brickwork. The transverse precast frames of the structure consist of two ferroconcrete uprights, connected by crosspieces at a height of 6.5 m over the heads of the rails. The junction plate of the crosspiece and the upright is located in the zone of the least bending moments. The supporting parts of crosspiece are presented as boxes of metallic sheets 10 mm in thickness, welded during installation to the matching metallic sheets on the butt end of the cantilever part of the column.

The structure is characterized by the following indexes, based on a 1 t capacity of the hopper.

The expenditure of precast reinforced concrete of grade 200 (with steel framework to the amount of 100-220 kg/m^3 of concrete) constitutes about 0.11 m^3, the expenditure of monolithic reinforced concrete equal to 0.13 m^3, the overall expenditure of reinforced concrete is 0.24 m^3. The weight of the steel structures and linings – 0.07 t. The overall weight of the supporting ferroconcrete and steel structures is 0.7 t, the same, allowing for the weight of a wall filling, is 0.99 t.

In Table 33 the indices of ferroconcrete hoppers having a 1 t capacity are given. A comparison is made only for ferroconcrete loading hoppers and those with a parabolic capacitive compartment. In this case the height of the hoppers with a parabolic capacitive part constitutes 15.0-15.4 m to the floor of the hopper gallery, and the height of the all-reinforced hoppers is equal to 18.2-19.5 m.

By comparing the indices of the ferroconcrete hoppers with the parabolic capacitive part and all-reinforced concrete hoppers, it is possible to draw the following conclusions.

Ferroconcrete hoppers with a parabolic capacitive part are characterized by a less overall expenditure of metal per ton of capacity of the hopper. The expenditure of reinforced concrete and the weight of the hoppers with a parabolic capacitive part are two times less than the corresponding indices for all-reinforced concrete hoppers. Furthermore, one ought to show that the number of type and dimensions of sectional ferroconcrete structures for hoppers with parabolic capacity is comparatively small; this determines the corresponding favorable indices during the manufacture and installation of these structures. The average cost of the support and enclosing designs for hoppers with a parabolic capacitive part constitutes about 80% of the corresponding cost for all-reinforced concrete hoppers. On the basis of that proposed, it is possible to draw the conclusion about the expediency of the wide utilization of section ferroconcrete loading hoppers with a parabolic capacitive part.

Table 33.

Designation of the indices	Unit of measurement	All-reinforced concrete hopper			Hopper with a parabolic capacitive part		
		according to Fig. 276	according to Fig. 278	according to Fig. 280	according to Fig. 282	according to Fig. 286	with suspentions at the bottom of the beams
Expenditure of reinforced concrete sectional and monolithic.	м^{3}/t	0,67 (0,55)	0,60	0,50	0,22	0,24	0,30
Weight of the steel structures and lining..................	t/t	0,05 (0,01)	0,06	0,04	0,06 (0,07)	0,07	0,06
Weight of the steel framework of the ferroconcrete structures	»	0,07 (0,05)	0,07	0,03	0,04	0,04	0,04
Overall investment of steel....	»	0,12 (0,07)	0,13	0,12	0,10	0,11	0,10
Weight of the reinforced concrete, steel structures and linings......................	»	1,67 (1,52)	1,63	1,34	0,63	0,70	0,84
Weight of the brick and slag-block laying..................	»	0,24 (0,20)	0,18	0,30	—	0,29	0,13
Weight of the lightweight wall barriers.....................	»	—	0,05	—	0,06	—	0,08
Weight of the supporting structures and wall barriers........	»	2,11 (1,72)	1,86	1,64	0,69	0,99	1,05
The same, with the replacement of the laying by a lightweight wall barrier................	»	1,95 (1,60)	1,75	1,45	0,70	0,80	0,95

Note: In brackets data are given not taking into account of the lining or incomplete lining, and also not taking into account of the stairs and shifting joints.

Hoppers with wooden and other designs. In small mines in a number of cases very simple loading hoppers are used (Fig. 287) with central loading. These constructions in a number of cases based on height correspond to the deckhouse buildings, set on higher sites. The deckhouse building in this instance is usually a one-story with haulage at ground level, sometimes – artifically created because of dumps of an earthen dam. The simplicity of the overall scheme adjacent to the mine shaft of the structure offers the possibility to have simple hoppers. Under conditions of nonheated structures, the central ore loading and the special equipment of a bottom make the described hopper suitable for operation under wintry conditions.

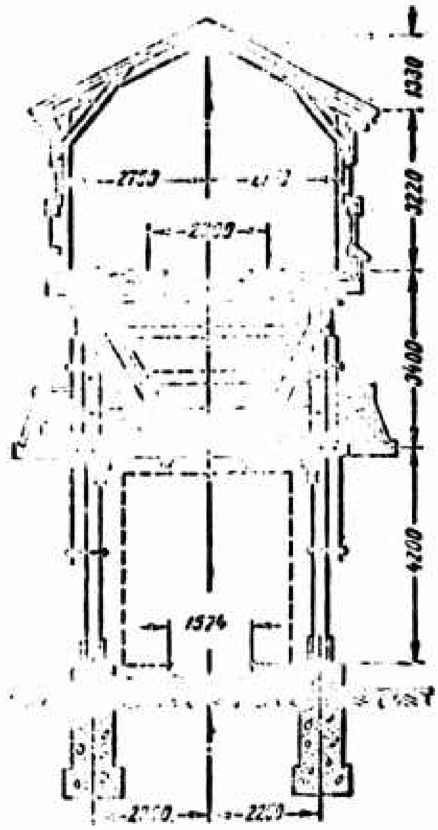

Fig. 287, The simplest wooden loading hopper with central loading.

Figure 288 gives the details of the transverse frame of the described hopper with a central loading. As it was noted the equipment for the bottom, assembled on a separate plank, is the characteristic feature. This type of hopper is unloaded into half-cars of standard gauge track (without including the locomotive)

or into motor vehicles and trolley carts under the condition of the successive removal of the plank in the bottom. For this it is only necessary to shift the outer plank along the hopper, not the support load from the ore, nor the arranged thinning out of the slope of the ore at the site towards one of the outpourings lengthwise to the construction. If the boards, located at the base of the slope of an ore, are gradually removed or simply shifted horizontally, then the ore will enter the positioned vessel under the hopper. In this case the slope of the ore will be gradually shifted along the hopper in the direction of the repeatedly removed board at the bottom.

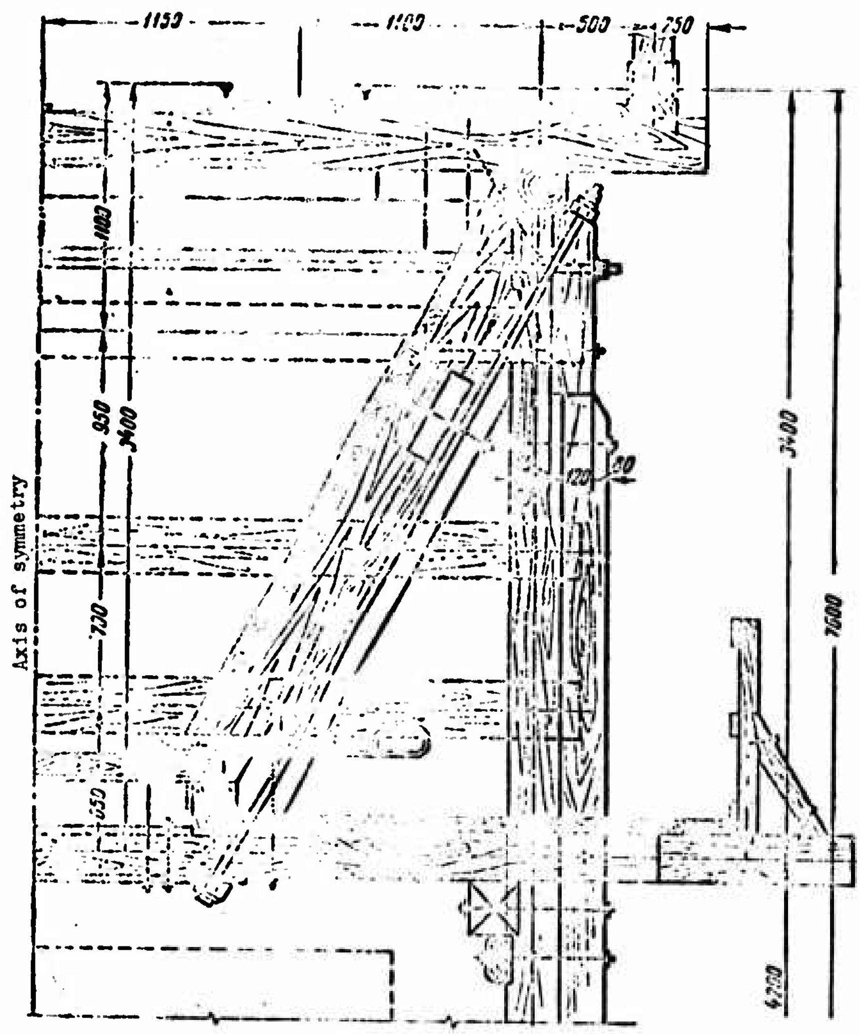

Fig. 288. The part of the transverse frame of a wooden hopper with central loading.

The feature of the design, given in Fig. 288, is the possibility of shifting the plank at the bottom from the outer platform, because of safety during the process of unloading the hopper.

The hopper according to Figs. 287 and 288 differs by a small height. In connection with this, the capacity of such a storage body is relatively small, and the indices of the expenditure of material per unit weight of the ore, found in the body, are quite considerable. Specifically, the capacity of the described hopper constitutes 15 t per 1 m of length of the structure, the expenditure of wood in this matter is 0.3 m^3, metal is 55 kg, concrete or rubble concrete is 0.35 m^3 per 1 t of ore.

Figure 289 shows a wooden one-sided hopper with lateral ore loading. The cell of the hopper has sizes, 4 × 4 × 10.5 m (without a hopper gallery). The transverse frames are set at 1.4 m. Every 4 m the transverse frames are attached to connection 1, which together with the uprights, form the frame of the transverse dividing wall of the hopper. This wall has vertical and inclined sheathing, which facilitates an increase in the rigidity. The bottom of the hopper is made of planks 100 mm thick, resting on inclined crosspieces 2 of the frames of the structure. Hoppers of this type differ by a high degree of rigidity, by simple and reliable cuts. The comparatively large number of transverse frames, the distance between them which usually does not exceed 1.5 m lengthwise to the structure is somewhat of a deficiency.

Figure 290 illustrates a hopper of the same transverse overall size. The size of the cell of the hopper, 4 × 3 × 10.5. The transverse frames here are set at a distance of 3 m, the number of frames is substantially low because of the reinforcement of the bracings of the bottom, which are taken at a height of 200 mm.

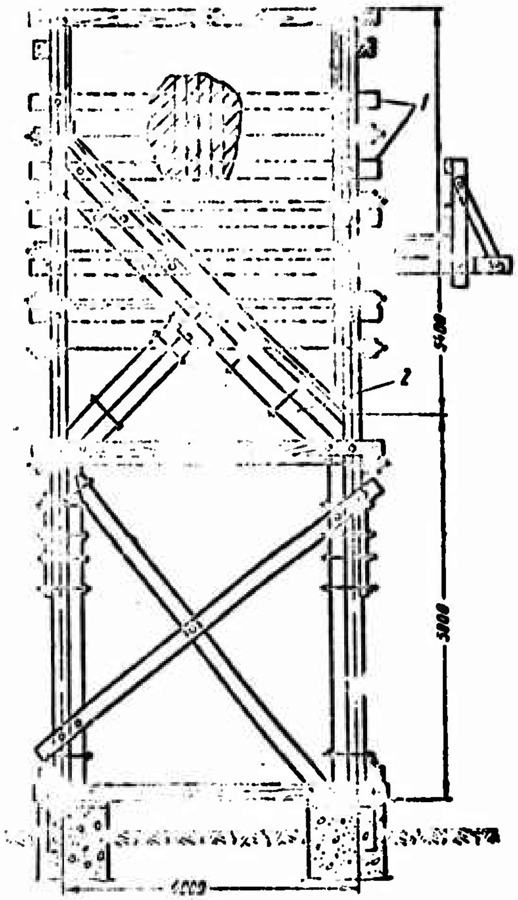

Fig. 289. One-sided wooden hopper with lateral ore loading.

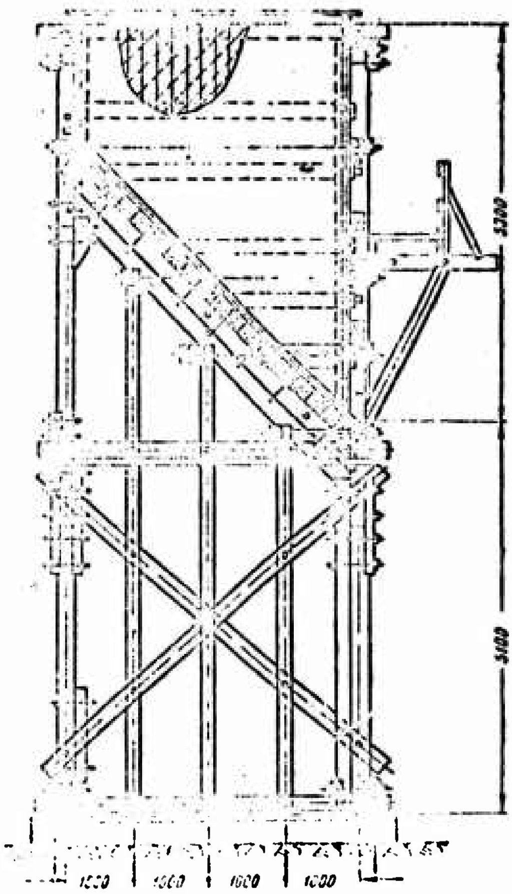

Fig. 290. One-sided wooden loading hopper with lateral ore loading along with a cell, 4 × 3 × 10.5 m, and transverse frames, spaced at 3 m.

That and other types of hoppers are repeated used in a number of enterprises. Below is given some comparison of a diagonal type of hopper with the size of the cell, 4 × 4 × 10.5, and a geometrical capacity of 40 m^3, or 100 t of ore (see Fig. 289), with a hopper of the erect type with the size of the cell, 4 × 3 × 10.5, and a capacity of 30 m^3, or 75 t (see Fig. 290). The indices of the expenditure of material of both cells are quite close. With the volumetric weight of the ore at about 2.5 t/m^3 the expenditure of wood in this matter constitutes about 0.2 m^3, the expenditure of metal, 50-60 kg, and with nonmetallic lining of the vertical walls, 20-30 kg, the expenditure of concrete 0.16 per 1 t of ore.

Diagonal hoppers with the size of the cell, 4 × 4 × 10.5, are characterized, relative to the large number of transverse frames, by a high degree of transverse rigidity; they are more advantageous with specified longitudinal supports, are more expedient with various combined and sectional ferroconcrete designs of the supporting part of the hopper, and more multipurpose in view of the larger number of transverse frames.

Upright hoppers with the size of a cell, 3 × 3 × 10.5, are simpler to manufacture, and they have a smaller number of transverse frames, but they require a provision of transverse rigidity.

As is known, hoppers sustain a series of forces, acting in a horizontal or inclined plane. Such forces are the forces of braking the rolling stock; the forces, which appear during the unloading and stopping of the latter; the action of different equipment, the possible horizontal reactions of inclined structures, the action of blasting and so on.

The noted advantage of hoppers of the upright type (Fig. 290) has, at the same time, a certain deficiency, since the reduction in the number of transverse frames leads to less rigidity of the construction crosswise, Therefore, transverse frames according to Fig. 290 should be rather rigid in their plane, whereby it is

necessary to firmly fasten the bracings to the uprights with the help of notches and several bolts of a diameter of not less than 24 mm. Drawbars for the inclusion of bracings in the work should be also installed in the frame. A section of bracings should be not less than 200 cm^2.

Figure 291 shows a double-sided hopper with lateral loading along with double cells, 2 × 3 × 3 × 10.5, of a geometrical capacity of 50 m^3, of 125 t.

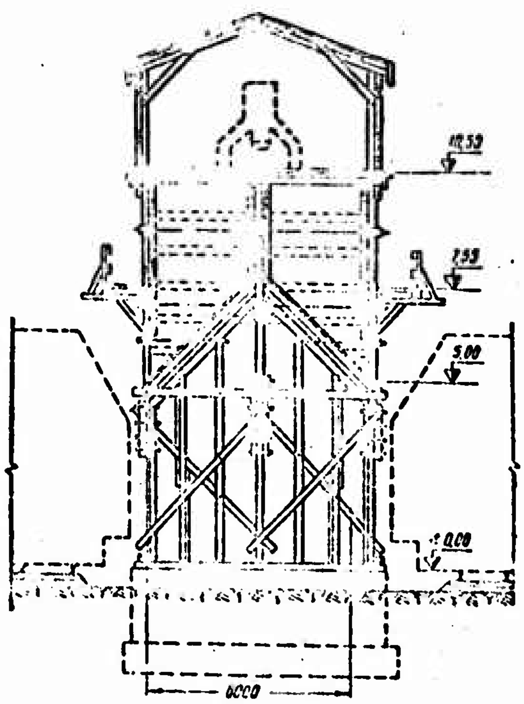

Fig. 291. Double-sided wooden loading hopper with lateral loading of loose material.

Figure 292 shows a wooden hopper with central loading into motor vehicles during their transverse movement. Despite the comparatively low height of the construction, equal to 9 m, the capacity of the hopper constitutes about 30 t per 1 m of its length. The horizontal rigidity of the hopper in a longitudinal direction is provided for by joints in the adjoining structures of hopper to the very rigid ferroconcrete housing of sorters. From the opposite end of hopper bracings are provided. The spacing of the uprights of the hopper is 4 m.

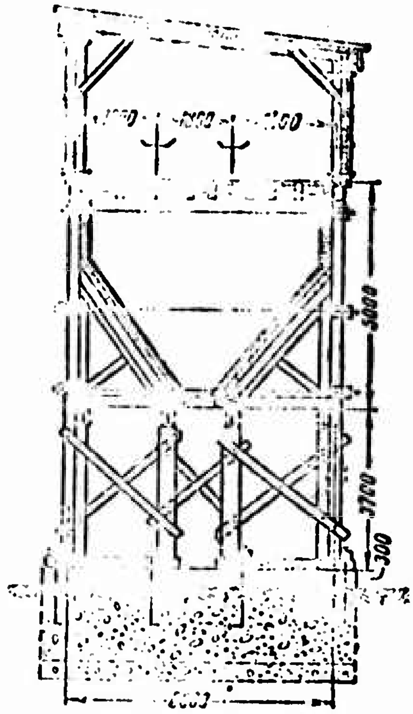

Fig. 292. Wooden hopper with central loading into motor vehicles along with their transverse movement.

It is necessary to show that the parts of the wooden loading hoppers are found under various conditions of operation. In the most complex upper part of the hopper, during the entry of relatively dry ore or flux, wood can be used under satisfactory conditions. If the hopper is provided with lining, which is regularly inspected and repaired during operation, the mechanical damages of the support structures are eliminated (under conditions of normal operation). The wood of the upper part of the hopper is usually uncovered at the bottom and covered towards the top. If the projection of the cornices is sufficiently great, and the horizontal uprights, at determined intervals, have return flow, then allowing for the above enumerated conditions it can be considered that the wood of the upper part of the hopper can be preserved over a rather prolonged period of time.

The hoppers with rot-resistant wood can be found in many old mine pits. The operation of these structures has long since ended, but protected wood is found practically in satisfactory condition. The lower part of such hoppers is subject to rot and damaged to a

height up to 1-1.5 m from the level of planning. The great height of the damaged section of the wooden structures of the hoppers can usually be explained by the specific conditions of operation. One ought to keep in mind that in the process of a loading or because of a defect of the lining and elements the bottom part of the loose material lies at the base of the uprights of the structure. Therefore, the uprights of the hoppers can always be covered. Some of these structures are filled over a long period up to the capacitive part, and only the danger of an unavoidable accident necessitates. an effort toward productive work for the cleaning and repair of the uprights and bracings, which, furthermore, are hardly protected, and are subject to the direct action of atmospheric moisture.

In this way, it may be concluded, that the diagram of the hopper in Fig. 293a is not perfect based on the conditions of operation. Such construction will exist over a comparatively short period, whereupon an overhaul of the hopper will be in order.

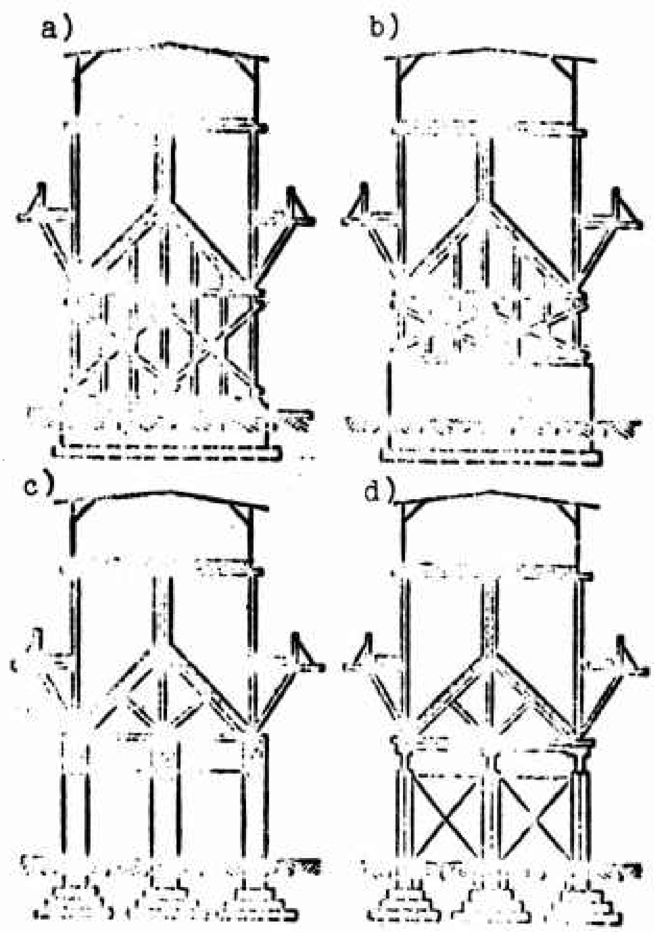

Fig. 293. The diagrams of double-sided hoppers with lateral unloading: a) diagram of a wooden hopper; b) diagram of a hopper with wooden structures having a height of 1.0-1.5 m over the level of planning; c), d) hoppers of complex structure with the equipment of the lower part of the construction made from monolithic or precast reinforced concrete with wood in the upper part.

It has been proposed to have the wooden structures of the hoppers at a height of not less than 1.5-1.0 m over the level of planning, as shown in Fig. 293b. In this instance the above described phenomena will be, to a considerable degree, averted. The absolute exception of the danger of damage to the wood of the hopper can be attained upon the completion of the lower structure of the construction with ferroconcrete. The corresponding diagrams of hoppers with lateral loading are given in Fig. 293c, d. The first of these diagrams corresponds to the equipment of the lower part of the hopper in monolithic reinforced concrete, and the second – in the precast reinforced concrete. The spacing of the uprights in the longitudinal direction in both cases can be equal to 6 m, the mutual distance of the uprights is 3 m. According to the diagram 2 the transverse frame is represented by ferroconcrete sectional uprights and metallic joints. Three longitudinal ferroconcrete sectional beams, set directly on the uprights, support the upper structure in each cell, made in the form of diagonal wooden hoppers with double cells having a capacity of 50 m^3 and more.

Figure 294 shows a hopper, made on one of the remote mine pits approximately according to diagram c (Fig. 293), i.e., according to the diagram using the monolithic ferroconcrete designs of the lower part of the hopper. A certain distinction of the described hopper with the recommended diagram is the equipment of the capacitive part using a cell not of the diagonal type, but of the upright type, which resulted in the need to introduce intermediate transverse beams every 3 m, resulted in the complication of the ferroconcrete part of the hopper and to a certain worsening of the indexes of the expenditure of material. Nevertheless, despite the noted deficiency, the indexes of the design, presented in Fig. 294, are rather satisfactory. Thus, with a volumetric weight of the ore of 2.5-2.0 t/m^3, the expenditure of wood in this matter constitutes about 0.15-0.18 m^3, the metal, 40-50 kg, and with nonmetallic lining of the vèrtical walls, 20 kg, the expenditure of reinforced concrete of 0.08-0.11, on the average 0.1 m^3 per 1 t of ore in the bunker.

The substantial circumstance, in this case, is the absence of the need to use a considerable volume of concrete or rubble concrete, applied under usual conditions of wooden hoppers as footing and other foundations.

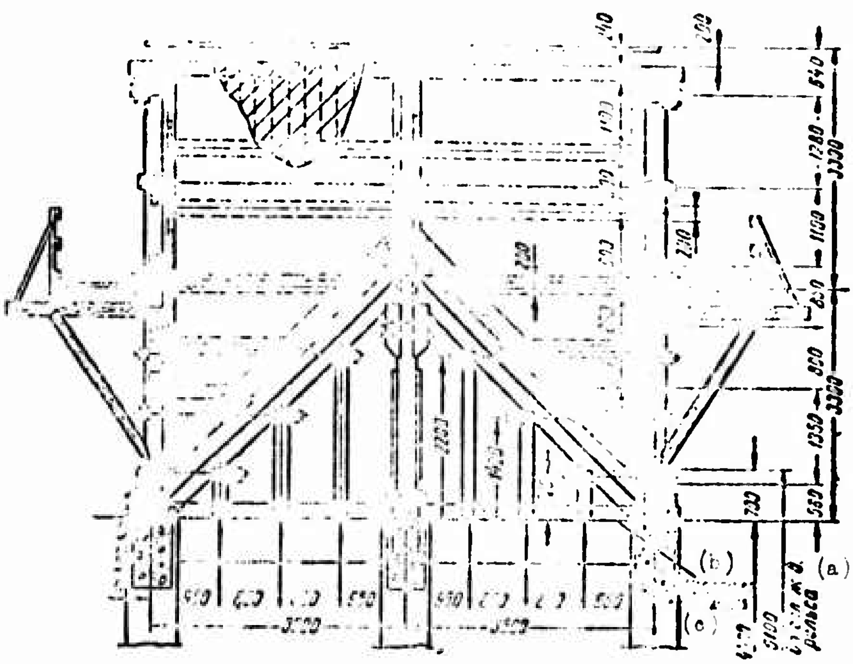

Fig. 294. A loading double-sided hopper of a combined design.

KEY: (a) up to the railhead; (b) insulation; (c) мол6 - illegible.

At existing prices the cost of hoppers according to diagrams, given in Figs. 293c, d and 294, is approximately equal to the cost of the all-wood loading hoppers, shown in Fig. 293a. However, the construction, completed according to scheme a, c (Fig. 293), are incomparable based on the capitol investment. Under the condition of the cover of wood from the atmospheric effects the hoppers, made according to the diagrams, shown in Figs. 293c, d, and 294, can remain in operation over a very considerable period of time.

Figure 295 gives one of the rational diagrams of the double-sided hoppers of a combined design with lateral loading. The grating of the ferroconcrete sectional uprights of construction - 6 × 6 or 7 × 6 m, the overall height of the construction, 10.5-12.0 m. The

cells of the capacitive part of the hopper have sizes by design $2 \times 3 \times 3$ or $2 \times 3.5 \times 3$ m. The sectional uprights having a weight of about 5 t are set in ferroconcrete shoes of the bootleg type and unitized by the usual means. Furthermore, the uprights are equipped with cross metallic joints in transverse and longitudinal directions, the presence of which substantially simplifies the solution of providing stability and strength to a hopper under the condition of blasting effects. Sectional ferroconcrete longitudinal beams 2 rest directly on the uprights 1 with welded bracing of the metallic backing parts. The frames of the capacitive part are formed by crosspieces, uprights and by delta-shaped wooden trusses 3, set on beams 2 spaced 1-3 m apart depending on the available selection of wood. As usual, horizontal tongs and drawbars on the junction plates of the cell spaced 3 m apart, are installed lengthwise to the structure. Trusses 3 supporting considerable loads, are made using metallic drawbars in the lower strap and central upright.

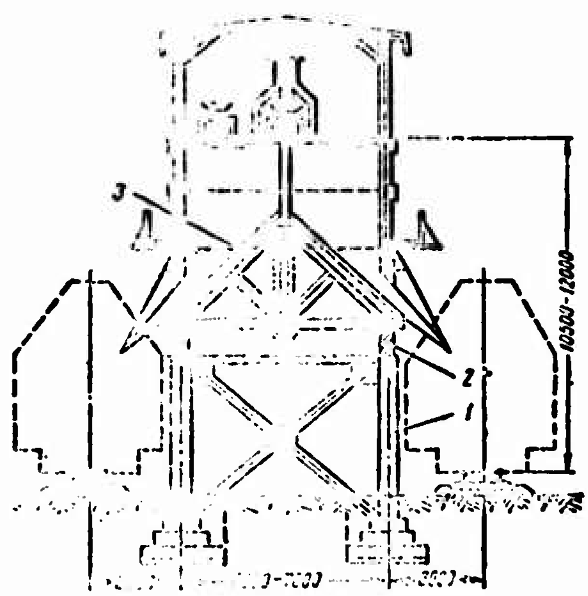

Fig. 295. The diagram of a wooden-reinforced concrete double-sided hopper with lateral unloading.

The described design of the sectional hoppers is characterized by the utilization of one type and dimension of ferroconcrete uprights and of one type and dimension of sectional girders.

The diagram of a hopper with lateral loading, given in Fig. 295, is also an expedient contemporary diagram under the condition of the fulfillment of the capacitive part of the hopper of metallic, trusses composed of steel 3. One ought to note, that in the case of erecting steel and ferroconcrete designs of the conventional improved hoppers with central loading in the technological relationship along with the introduction of transport vessels under the capacitive part (Fig. 282 and others).

4. The Protection of Structures from Abrasion and Blows

Above mentioned examples indicate the need for the lining of the receiving funnels and corresponding shockproofing.

One can consider that the thickness of steel lining depends, above all, upon the volumetric weight and the coarseness of the ore, of load-carrying capacity of the transport or lift vessel, and the productivity of the undertaking. During the unloading of fine crushed ores and with the load-carrying capacities of the transport vessel up to 5 t, the thickness of the lining is taken equal to 8-12 mm. With an ore of average lumpiness and with vessels having a load-carrying capacity up to 10 t, the thickness of the lining is 16-20 mm. During the unloading of lumpy ore from the dump cars and heavy dump trucks, a lining of a thickness of 30-50 mm is used. For general lining predominantly rolling sheet steel of various grades is used. For the most severly worn out sections, which are found in the path of continuous flow of ore, in the places of the spill, unloading, discharging of the ore, the utilization of abrasive-resistant managanous high-carbon steel of 30G2 and other grades is necessary. In a number of cases steel of St. 5 grade is used for lining. On sections, where accelerated wear is not expected, steel of St. 3 grade and others are used for lining.

In all cases the protection of the welded seams against blows and for the alignment of seams, is highly important. The latter are usually arranged parallel to the flow of falling ore. In this instance, the flank seams work on the main sustaining structures of the load in order that the straps of band steel can be used as a cover (see Fig. 265c, d).

With the unloading of the lift vessels from the shafts or with fine particle ore the thickness of the linings diminishes. The thickness of the lining of the vertical walls, less subjected to blows and abrasion, other conditions being equal, is taken 50-30% less than the thickness of an armor shield. With very heavy lumps, which fall from a great height, the lining along the appropriate planes are reinforced in order to increase its thickness, increase the abrasive-resistance and the utilization of special designs.

In the receiving hoppers with steep slopes of planes, the lining is made from the rails of the standard gauge track, set parallel to the motion of the ore (see Fig. 273). In this instance towards the top rails are set on the comparatively thin sheets of a thickness of 10 mm, spaced every 200 mm, and welded with the railfoot on the sheet metal. In certain cases towards the top where the sheets are parallel to the slope, bands of abrasive-resistant steel are laid down having a thickness of 40-80 mm, fixed to detachable sheets with bolts. During the repair of the linings one replaces a part of the bands and sheets with armored bands, and again it is fastened with bolts to the elements of the receiving funnel.

In hoppers and storage bins of the ore treatment plants, which are located in the main flow of ore processing, detachable sheets of high-carbon steel and castings of manganous steel plates are used, having an area of 2-3 m^2 (Fig. 296). These plates are fastened with the help of six bolts; using an ore with the sizes of lumps, 600 mm and more, they are installed on the shockproof padding of the uprights.

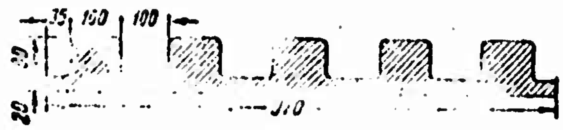

Fig. 296. A section of a cast lining plate with a complex surface.

With strong flows of comparatively fine particle-size ore, in all cases of expected wear of the lining, because of severe abrasion, one should use abrasive-resistant lining from hornblendite plates and other rock castings (Fig. 297d).

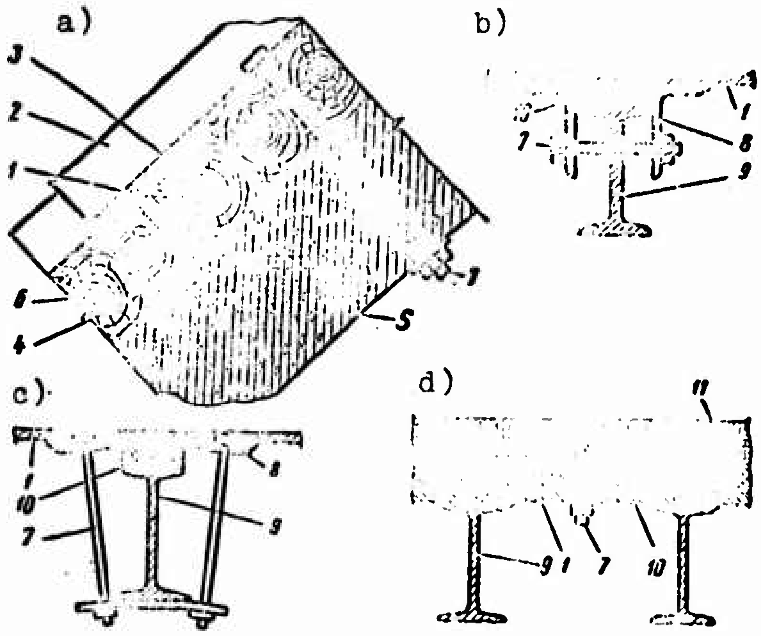

Fig. 297. The lining of the receiving funnels: a) part of the lining from sectional metallic shields; b), c) parts of the detachable lining; d) lining of the abrasive-resistant plates; 1 – the sheets of the lining; 2 – rails; 3 – corners; 4 – angle seat; 5 – ferro-concrete bottom; 6 – shockproof layer made from wooden braces; 7 – bolts; 8 – segments of angles, specified for sheets of lining; 9 – elements of the support design of the funnel; 10 – rubber and other padding for shockproofing; 11 – lining made from a rock casting.

At one of the enterprises the lining of the funnel is made of sectional metallic shields having a length up to 5 m on the slope with a width up to 2.5 m. The shields (Fig. 297a) are formed by

sheets 1 with welded on them rails every 160 mm 2; below the sheets, across these rails, horizontally corners 3 having a section, 200 × 20 mm are located and they transmit the acting forces along the slope into the ferroconcrete bottom 5 to the anchored angle seat 4. Between the steel sheets 1 and ferroconcrete structures 5 located a shockproof layer 6 made of wooden uprights is placed. The shields of the lining are tightened by the bottom with bolts 7 through gas pipes, put in the ferroconcrete slab.

There is also a number of other structures of sectional lining and shockproofing. All the designs guarantee the possibility of replacement or repair of any section of the lining.

In the larger funnels the warming of the bottoms is frequently done, the effectiveness of which is assured by the arrangement of the steam pipes above the supporting structures. Thus, for instance, in the described receiving funnel (see Fig. 273) the heating pipes are located between the rails and are covered by sheets of steel.

In the lighter receiving funnels (of the type given in Fig. 269), calculated for receiving fine particle ores, detachable lining is also used. In this case let us turn our attention to the stability of the frame of the funnel which, in this instance should possess sufficient rigidity and immutability, without taking into account the steel sheets, which here are only the lining. One kind of solution is given in Fig. 297b. The fastening bolts 7 are installed with the reduction of the elastic padding 10; in a number of cases, the fastening can be preliminarily made to one upper bolt (inclined section), whereupon the lining of the funnel is preloaded. In this position with the pressed layer of shockproofing 10 the bolts 7 are installed. Figure 297c gives more convenient variation of the bracings of the sectional lining.

In many instances the need appears for lightweight receiving hoppers, sustaining an impact pressure during the unloading of the lift vessel and other vessels from an ore of average coraseness and

fineness; the planes of the receiving hopper in this case can be arranged thus, since this is shown in Fig. 297c, and serving as lining sheet alloy steel, cast-iron plates and other material are used. Figure 297c, specifically, shows the abrasive-resistant lining with the utilization of rock casting.

Control gratings on the receiving hoppers for lumpy ore, just as for lining, sustain blows from the falling ore and in an absence of shockproofing the gratings are subject to especially unfavorable conditions; which is confirmed by actual operation of the receiving hoppers. During the action of the falling lumpy ore, the direction of blow is not always strictly vertical. Moreover, in view of the eccentricity of the blow, there are always considerable horizontal component forces of the blow. As a result of this the beams of the grating undergo considerable torsion. The process of accretion of the deformations develops extremely rapidly because of the simultaneous increase of the component forces of the blow, acting in a plane of the flange of the beam. With the accretion of deformations of the torsion, even a central vertically directed blow in relationship to the beam, can lead to the failure of the beam. In a number of control gratings above the receiving hoppers frequently various deformations appear. Apart from that described above, here local deformations of bending of the upper and lower flange of the double-T beams, the all possible forms of failure of the welded beams, specifically, the bending of the flange, the knocking out of the diaphragm, failure of the seams, and so forth, are usually observed.

In practice, various methods of reinforcement of the supporting elements of the control grating are used. Thus, for instance, double-T beams are tightened by transverse bolts, drawbars, arranged in pipe-spreaders and so on. These measures usually do not give apparent results. The introduction of vertical sheets 2 (Fig. 298a),

arranged symmetrically relative to the axis of the double-T beam 1, and which facilitate the creation primarily the entire enclosed section gives somewhat better results, more rational when working under torsion. Furthermore, in this instance the protection of the upper and lower flange is assured against transverse bending under impact.

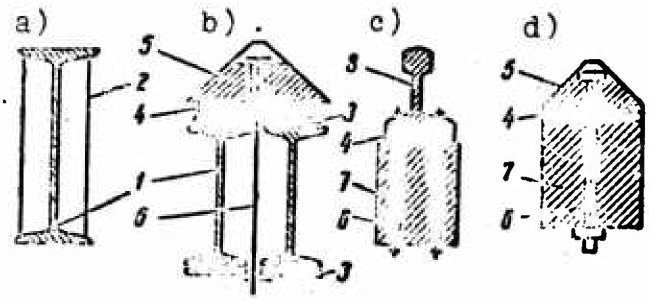

Fig. 298. Parts of the control gratings: a) reinforcement of the double-T beam of the control grating by vertical sheets; b) section of the elements of a control grating made from rolled beams or channel bars; c), d) variants of sections of the elements of the control grating; 1 – beam; 2 – vertical sheet; 3 – sheets; 4 – shockproof padding; 5)– protected steel casting; 6 – vertical bolt, gib-headed in the cut of the casting 5; 7 – billet (a bloom); 8 – rail.

It is obvious that under these conditions an enclosed or continuous section is more useful. However, the sections, as a rule, should not be welded.

The required section can consist of channel bars with flanges on the inside or of double-T beams with cut external sections of the flange on the largest part of the lower strap (Fig. 298b). In a number of cases the sections, presented in Fig. 298c, d, can be used.

Another solution of the control grating is given in Fig. 299. Here, cast steel beams and spacers of special profile are used. The beams are tightened by transverse belts, arranged inside the rigid spacer, set on the small ribs of the beams. Only these ribs and ends of the spacer undergo mechanical treatment. In all cases

the same type of grating, made in the form of joined castings are expedient in the composition of two and more beams with transverse butt-end elements.

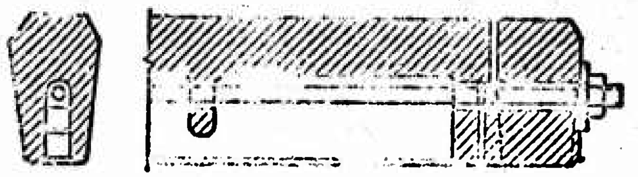

Fig. 299. Control grating made from cast beams and spacers of a special profile.

For a rather complex receiving funnel it is natural to suspend small, and a more complex metallic funnel to the basic ferroconcrete funnel with relatively simple outlines. However, this metallic funnel can be stripped under the impact of pieces of ore. Therefore, even with the average coarseness of falling pieces and with capacity of a transport vessel within the limits of 10 t, one ought not to permit a suspension of the funnel according to diagram b, Fig. 300. Such a suspension can be employed only according to the diagram a Fig. 300, whereby the torn away bolts can be replaced. Furthermore, in the latter case the utilization of elastic construction at the points of bracing of the metallic funnel is necessary.

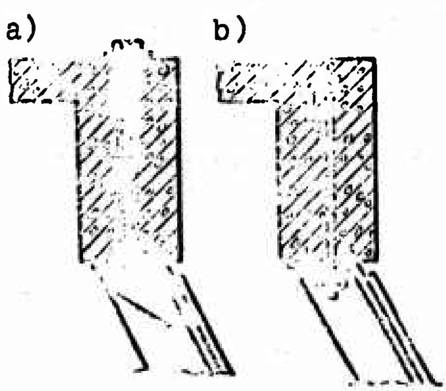

Fig. 300. The suspension of a receiving funnel or its lower part: a) correct; b) incorrect.

Another possible solution of the lower part of the receiving funnel is presented in Fig. 272, where the beams of the bracing of the lining are projected beyond the point of the ferroconcrete part

of the funnel. The metallic beams and sheet metal of the funnels here somewhat resist the impact pressures. The detachment of the lower part of the funnel in this case is excluded.

The lining of the loading hoppers, described above, is simpler and lightweight in comparison with that given here. With the filling of the loading hopper using conveyors, the thickness of the steel lining of the bottom of the hopper can be taken as 6-8, 10-12 mm, and the walls - 4-5 and 6-8 mm. The first values correspond to the short period of operation, small dimensions of the construction, lightweight features, wooden structures, but intermediate values - to the normal period of operation, to the various combined designs described above.

The last values can be taken over a long period of operation of the capital structures of considerable capacity.

Just as in the earlier described cases, the lining of the loading hoppers should have a design, which assures easy replacement in proportion to the wear of individual sheets. For this purpose the sheets of lining are suspended in their upper part on bolts, hooks an spikes, and the heads of bolts and other fastening parts are covered by the lower part located higher than the sheets of the lining.

In a number of cases the wear of lining in the receiving hoppers and in separate joints of the loading hoppers in various troughs and points reaches considerable proportions. Individual cases are known of the abrasion in joints with the continuous movement of the ore, by 2 mm per month. The economy of the metal in the spillways, trough, funnels and hoppers is, in itself, an important problem.

Prevention of abrasion in the receiving hoppers should be handled in a special way, by using abrasive-resistant steel; by means of hardening the steel and by various surface treatment. The

utilization of special steel in hoppers is possible in the form of special cast plates, and also in the form of square rods, band metal and other metal, exposed to the direction of movement of ore above the sheet backing. One of the simplest solutions is the above described lining with rails installed above it by thermally treated heads. That which is shown should also pertain to troughs and shifting joints with the motion relatively to the lumpy ore.

As for the individual joints, the loading of the hoppers and various troughs with the movement of the material having a coarseness up to 20 mm, in a number of cases can also warrant the use of substitutes of metal, for example, sheet glass of a thickness of 8-12 mm or more, simple and reinforced nonmetallic casting.

Sheet steel can frequently be economized by other ways as well. Thus, in the loading hoppers of the two mentioned groups, in particular, in hoppers with a comparatively short as well as the normal period of operation, in wooden hoppers and hoppers of a combined design, there is the considerable number of transverse walls. The lining of these walls constitutes a large part of the entire lining of the hopper. In a conventional double-sided wooden loading hopper with lateral unloading of loose material (see Fig. 291) with cells, 3 × 3, by design the area of the lining of the bottom constitutes 11.5 m^2, and the area of the lining of the walls of the same cell – 41.5 m^2, 78% of the overall area of lining. At the same time in the walls of the hopper the utilization of various substitutes of metal, specifically wood is easy to do. The utilization of wood for this purpose in forested and remote regions is entirely expedient and feasible for all usual fractions of an ore. The utilization of such lining of vertical walls hardly reflects the conditions of the movement of an ore under the hopper, especially during the partial utilization of metal under the lower part of the walls. In this instance in a rather conventional loading hopper of 15 double cells, it is possible to reduce the expenditure of metal to lining on the average by 50 t.

5. The Calculation of Ore Hoppers

Loads. The walls of the hopper - usually vertical or inclined. The bottoms of the hopper are inclined toward a horizon at angles of 50, 55, 60°. Lesser values of the angle of the slope of a bottom correspond to prismatic hoppers, the large values - to pyramidal hoppers. In a number of cases the bottoms are taken as horizontal and the movement of loose material proceeds under conditions of the movement of the ore over ore.

The chief loads on the hoppers are: the weight of the loose material, corresponding to the pressure on the walls of the hopper, the impact action of the falling material on the hopper (funnel) and the effect of blasting.

Vertical pressure in a hopper (Fig. 301)

$$P_{в} = \gamma h;$$

horizontal pressure

$$P_{г} = \left[\operatorname{tg}^2\left(45 - \frac{\varphi}{2}\right)\right]\gamma h = k\gamma h = kP_{в},$$

where γ - volumetric weight in the saturated state, t/m^3; h - distance along a vertical line from the point, where the pressure is determined, to the top of the hopper, m; ϕ - the angle of internal friction;

$$k = \operatorname{tg}^2\left(45 - \frac{\varphi}{2}\right).$$

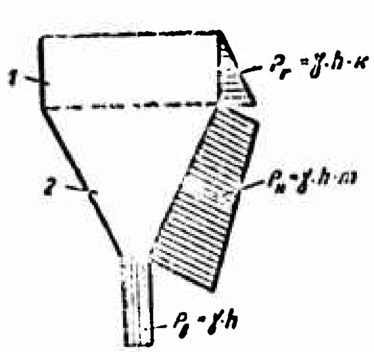

Fig. 301. A diagram for the determination of the pressure on the walls of the hopper: 1 - prismatic part; 2 - pyramidal part (funnel).

The angle of internal friction ϕ for iron ore fluctuates within the limits of 40°30'-51°30' and predominantly constitutes 45-47°. The same angle for crushed limestone based on different datum is close to the value, 40-38°. For other various loose material the angle of internal friction is frequently approximately equal to the angle of repose and somewhat less than the latter. Therefore, in a number of cases the value ϕ is taken tentatively equal to the angle of repose.

The volumetric weight of various loose materials, accumulated in mine hoppers, is given below:

<u>Limestones</u>

Limestone, crushed (0-25 mm).................	1.35-1.4
The same, after densification................	1.4-1.7

<u>Rocks and Tailings</u>

Waste rock from the quarry...................	1.6-1.9
Tailings, small (0-10 mm), iron content, 10%..	1.8-1.9
Bouldery ore. Manganic ore..................	1.5-2.0

<u>Agglomerate</u>

Agglomerate, blast-furnace and martensitic....	1.6-2.0

<u>Iron Ore</u>

Magnetite from a quarry and underground workings, iron content up to 40%.............	2.1-2.5
The same, iron content, 50%..................	2.5-2.7
The same, iron content, 60%..................	2.8-3.0
The same, iron content, more than 60%........	3,0-3.5

Wasned Iron Ore

Martite for blast-furnace melts from the open pits of washing plants, iron content of 50%.. 2.4

Martite for blast-furnace and martensite melts, iron content of 50-60%................. 2.5

Martite for martensite melts (10-300 mm), iron content of 65%........................... 2.9-3.0

Iron Concentrate

Fine and coarse dry concentrate, iron content of 25-35%.................................... 1.9-2.3

Dry and wet concentrate, iron content of 55-60%.. 2.4-2.8

Copper Ore

Copper ore, containing pyrites, after the first stage of rushing to 200 mm.................... 1.7-2.0

Copper ore, containing pyrites................ 2.5-3.0

Copper Concentrate

Copper concentrate, copper content of 20-25%... 3.0-3.5

Lead concentrate.............................. 3.5-4.0

Normal pressure $P_{\text{н}}$ to the inclined bottom of the hopper, set at an angle towards the horizon,

$$P_{\text{н}} = P_{\text{г}} \sin^2\alpha + P_{\text{в}} \cdot \cos^2\alpha = P_{\text{в}} k \sin^2\alpha + P_{\text{в}} \cos^2\alpha =$$
$$= P_{\text{в}} (k \sin^2\alpha + \cos^2\alpha) = P_{\text{в}} n = \gamma h n,$$

where

$$n = k \sin^2\alpha + \cos^2\alpha.$$

The values of m are given in Table 34.

Table 34.

α	The values at the angles of internal friction φ				
	35°	38°	40°	45°	50°
44	0,618	0,632	0,622	0,600	0,581
46	0,629	0,606	0,595	0,571	0,555
48	0,598	0,580	0,568	0,523	0,525
50	0,572	0,553	0,540	0,513	0,495
52	0,547	0,527	0,514	0,486	0,466
54	0,522	0,501	0,487	0,457	0,437
56	0,500	0,477	0,462	0,430	0,409
58	0,476	0,452	0,437	0,403	0,382
60	0,453	0,429	0,413	0,378	0,355

The pressure along the slope

$$Pk = \gamma h(1 - k)\cos\alpha\sin\alpha.$$

One ought to note that the relatively frequently determined values of normal pressure $P_{\text{п}}$ at frequently accepted inclines of the bottom 45-50° and the angles of internal friction within the same limits constitute 60-50% for an iron ore from the value of the vertical pressure $P_{\text{в}}$. Within these limits the normal pressure $P_{н} \approx 0{,}55\, P_{в}$, which is convenient for a tentative judgment about the strength of the elements of the inclined bottom of the hopper.

The impact action of a falling ore into the hopper can hardly be observed during the unloading of large-size transport vessels filled with lumpy ore, delivered into the receiving hoppers-funnel from the open pit mine. The fall of the mass of ore occurs from a height of 5-15 m. With a uniform supply of small loose material in the depository, the impact action of the ore can hardly be observed. In this way, the means of loading the transport vessels into the hoppers is highly important.

The effect of an impact load is especially strongly manifested in small hoppers, loaded with lumpy ore, and in separate parts of hoppers, directly sustaining the load. Such parts are the control gratings of hoppers, lining, the sheets of the funnels. Under severe

conditions the welded seams of the lining and bolts, then supporting suspension funnels, receiving the impact of the lumpy ore, all are subjected.

There is no theoretical data on the impact action of falling ore. In practice it is known that during this phenomenon it is frequently possible to observe the failure of any unprotected structure, and that in these cases, active measures for absorbing the shocks are necessary. Taking into account these preventive devices the value of the arbitrary dynamic coefficient in the calculation of the receiving funnels and hoppers taken equal to 1.2-1.5 m and in excluded cases, up to 2. For separate joints and bolts, which are subject to the most unsatisfactory conditions, this coefficient is taken equal to 2-3. The coefficients of overload are taken within the limits of 1.2-1.3.

The effect of blasting is determined in accordance with the earlier given indications. In this case, one considers that for the calculated value of seismicity, which corresponds to conducting mass explosions, all the containing bodies are predominantly empty, and that during the usual, relatively frequent, but small explosions, the seismicity scale is depressed by one or two units. The latter corresponds to a decrease in the seismic loads by 2 or 4 times, which is equivalent to a reduction in the weight of the loose material in the depository by as many times. The practical method of calculating the hoppers and a number of other ore and nonore depositories amounts to figuring out the calculated seismicity scale as usually 50%, and sometimes 25% of the weight of the loose material, which correspond to the complete geometrical filling of the hopper. This pertains primarily to the loading hoppers, which can always be emptied over a comparatively short period. Noted below are several explanations of the described method of calculation.

As stated in Chapter II, mine constructions are calculated under the assumption of the action of arbitrary seismic loads. In

this case the calculated values of seismic forces constitute considerable values. Thus, for a cell of the loading hopper, 6 × 6 m, at a height up to the level of the conveyor gallery of 15 m, the capacity of the depository is equal to 360 t. The expenditure of reinforced concrete in this instance constitutes about 0.6 t; the metal structures — about 0.1 t; other material — about 0.4 t; about 1 t (or somewhat more) per 1 t capacity of the hopper. Neglecting the values of the coefficient of dynamicity and the coefficient of form, the normative seismic load on the cell of a half-unit [semiball] the calculated seismicity will be 18-20 t. The normative windy load of the same cell under conditions of the first geographical region is equal to 4 t. In this case the seismic load exceeds the windy load by 5 times; consequently, the more precise seismic loads in the regions of blasting as applied to the loading of other hoppers has an especially substantial value. For the same reason there is the possibility to lower the rated value of the arbitrary seismic forces for the loading of other hoppers as well because of the improvement in the organization of the blasting.

Frequent blasts at mine pits are characterized, as a rule, by the weight of a discharge within the limits of several tons. Blasting explosions with discharges within the limits of 10 t is done much less frequently. Finally, mass blasting with the weight of the discharge up to 100 t and more, is done rarely, and the intervals between them are usually measured in months, even in years. The mass blasting is accompanied by relatively long, especially serious and many faceted preparation, which includes; as a rule, the emptying of all significant capacities and primarily, the loading hoppers. Furthermore, in a process of the preparation of mass blasting the degree of the safety of the existing structure and radii of seismically dangerous zones, are checked.

However, the ball-scale of the calculated seismicity is set namely allowing for the possibility of mass blasting, when the content of the capacities is predominantly lacking. From this viewpoint during the determination of the seismic forces of the

loading and other comparatively rapidly unloaded hoppers, one ought to introduce in the calculation only a certain part of the weight of the ore – usually 50%, but sometimes 25% of the weight of the loose material in a depository during its complete loading. Based on the designated figures it is possible to present the following.

The basic index, which characterizes the possibility of failure in buildings and construction during blasting, is the velocity of vibrations. The magnitude of the latter is proportional to the weight of the discharge to the 1/3 power. A substantial increase or reduction in the velocity of the vibrations means the corresponding change in the ball unit of seismicity. Thus, for instance, the velocity of vibrations during the blast of a discharge having a weight of 10 t, which is limiting under usual conditions, is 2.15 times less than the velocity of vibrations during the mass blasting with a discharge of 100 t. This is equivalent to the fact that the magnitude of the seismic load in the first case can be reduced by two times, and the calculated ball unit of seismicity – by a unit against the corresponding indexes during mass blasting.

In accordance with the given data the values of seismic loads for various cases can be characterized in the following manner.

Let the *first case* correspond to the moment of production of the mass blast and to the set calculated ball unit of seismicity. Let us designate: P – The weight of the ore in the hopper; G – the weight of construction without the ore; $K_c = \frac{1}{n}$ – seismic coefficient.

By introducing 50% of the weight of the ore into the calculation, as indicated above, neglecting the dynamic coefficient and coefficient of form, and taking into account the crude equality of the values of P and G, it is possible to obtain the seismic load, equal in this instance, to

$$K_c\left(G+\frac{P}{2}\right)\approx\frac{1}{n}\left(P+\frac{P}{2}\right)=1.5\frac{P}{n}.$$

In the *second case* during usual small blasting and with the reduced ball unit of seismicity, the value of the seismic coefficient

$$K_c'=\frac{1}{2}\,\frac{1}{n}=\frac{1}{2n},$$

but the value of the seismic load with 100% of the filling of the hoppers will be equal to

$$K_c^1(G+P)\approx\frac{1}{2n}2P\approx\frac{P}{n},$$

which composes only 2/3 of the above determined calculated seismic load of the hoppers.

Analogously, upon the introduction of 25% of the weight of the ore into the calculation during mass explosion based on the first case,

$$K_c\left(G+\frac{P}{4}\right)\approx\frac{1}{n}\left(P+\frac{P}{4}\right)\approx1.25\,\frac{P}{n}.$$

Here, the value of the seismic load with 100% of the filling of the hoppers and during conventional blasting will amount to 80% of the accepted load. Consequently, in this instance, along with the calculated load the determined reserve of the loads is taken into account, which compensates for some reason for the emptied section of capacity.

Based on the strength of the worst conditions of emptying capacities, when determining the values of the arbitrary seismic loads of the loading hoppers today in the regions of blasting it follows, as a rule, to consider 50% weight of the ore for rigid hoppers, which is equivalent to 350% of the windy load. The calculation of 25% of the weight of the ore for this purpose approximately corresponds to 250% of the windy load. The last amount (25% of the weight of the ore) should be taken in the calculation only for the

loading hoppers and other easily emptied hoppers which are located, relative to the most distant from blasting, (beyond $2r_c$) at the industrial sites of mines, and also calculated for loading hoppers with a flexible suspension (parabolic) of the capacitive part, located beyond r_c. Subsequently, when conducting the necessary observations, apparently, the wider utilization of the last means of determining the arbitrary seismic loads will be possible.

The location of the unloading of the hoppers during mass blasting should be clearly stated in drafts and instructions.

In a part of the intermediate hoppers of ore processing combines one ought to note that their complete emptying in certain cases is difficult. The reasons for the difficulties basically are the variable capacities and the different times for emptying the hoppers when their unloading is necessary into the circuit of the ore dressing apparatuses. Therefore, the problem about the magnitude of seismic forces for intermediate hoppers should be solved individually for each case in reference to the technological conditions and the arrangement of the construction relative to the bounds of the seismic dangerous zone during the blasting.

When calculating the hoppers and funnels, the coefficients of overload for the determination of pressures on the walls of the hopper are sometimes taken equal to 1.3. In a number of cases with accurate data on the value of the greatest volumetric weight of the ore after it has caked the coefficient of overload is taken equal to 1.2. With the loading of the hopper by conveyors, the dynamic action of the loads from the weight and pressure of the ores is not considered. Otherwise, the coefficient of dynamicity is taken according to the earlier given positions in relation to other construction. Under usual conditions of loading of the receiving funnels, the coefficient of dynamicity is taken to be not more than 1.3-1.5:

The calculation of rigid ferroconcrete hoppers. As a result of the calculation of the coefficient of overload and dynamicity the diagram consists of the calculated pressures, the character of which is similar to the diagram of normative pressures (Fig. 301). The facets of the funnels of the hopper are calculated for the following forms of reinforcement:

1) for the local bending from the load, normal to the slope;

2) for the tension in a horizontal direction from the reactions of adjacent facets;

3) for the rupture of the funnel;

4) for the general bending of the hopper as spatial box supported at angles.

Each facet is calculated for local bending under the action of the calculated load with the inclusion of the component of the weight of the wall. If the lower side a_1 of the facet is less one-quarter of the upper side a_2 (Fig. 302a), then the facet is calculated as a delta-shaped plate, and the reinforcements are determined according to the tables. If the lower side of the facet a_1 is more than 0.25 a_2, then the facet is considered as rectangular with given sizes for the calculation:

$$l_p = \frac{2}{3}(2a_1 + a_2)\frac{a_2}{a_1 + a_2};$$
$$h_p = H - \frac{a_2(a_2 - a_1)}{6(a_1 + a_2)}.$$

Under tension in a horizontal direction from the reactions of the adjacent facets P_1 (Fig. 302b), the facet is calculated as an eccentrically stretched facet allowing for the moments from local bending.

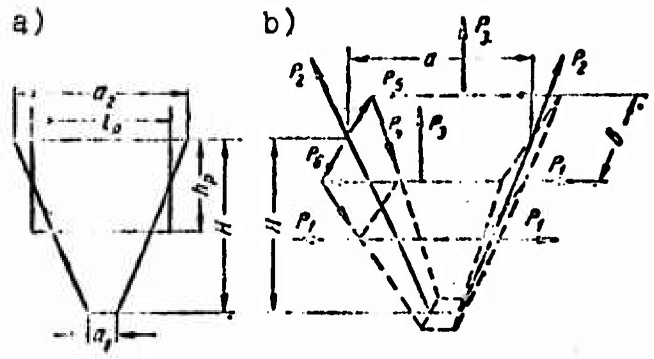

Fig. 302. The diagram of the funnel of the hopper: a) for the calculation of the facets during the action of a normal load; b) for the calculation of the funnel under tension and breaks.

The calculation of rupture of the funnels with resultant forces P_2 and P_3 having the support of hopper on columns, is done under the assumption that the concentration of tensile forces is in ribs. The rupturing force in the inclined rib P_4 is different from the vertical supporting reaction of the column taking into account the the angle of slope of the rib towards the horizon, $\frac{P_4}{\sin\alpha}$. The stretched steel framework should be concentrated in the ribs and firmly anchored on the supports. Simultaneously with the tension in the inclined rib, compressive forces P_5 and P_6, arise along the horizontal rib of the hopper which are determined during the examination of the equilibrium of the joint.

With support of the hoppers at angles on the columns of the structures, the hoppers undergo general bending in two directions. The corresponding bending moments are sustained by the wall beams by facets of the hopper. With a high prismatic part of the hopper, one takes into consideration the bending of the rectangular wall-beams, in pyramidal hopper-funnels – the bend of the delta-shaped wall-beams.

The diagrams of the reinforcement of the ferroconcrete hoppers are given in reference books and in other material.

The calculation of hoppers with a parabolic capacitive part. In practice the calculation of the designs of a parabolic capacitive part can amount to very simple calculated operations with a known form of a funicular curve. If curve ADF (Fig. 303) is known, then the calculation of the parabolic hopper can be made in the following manner.

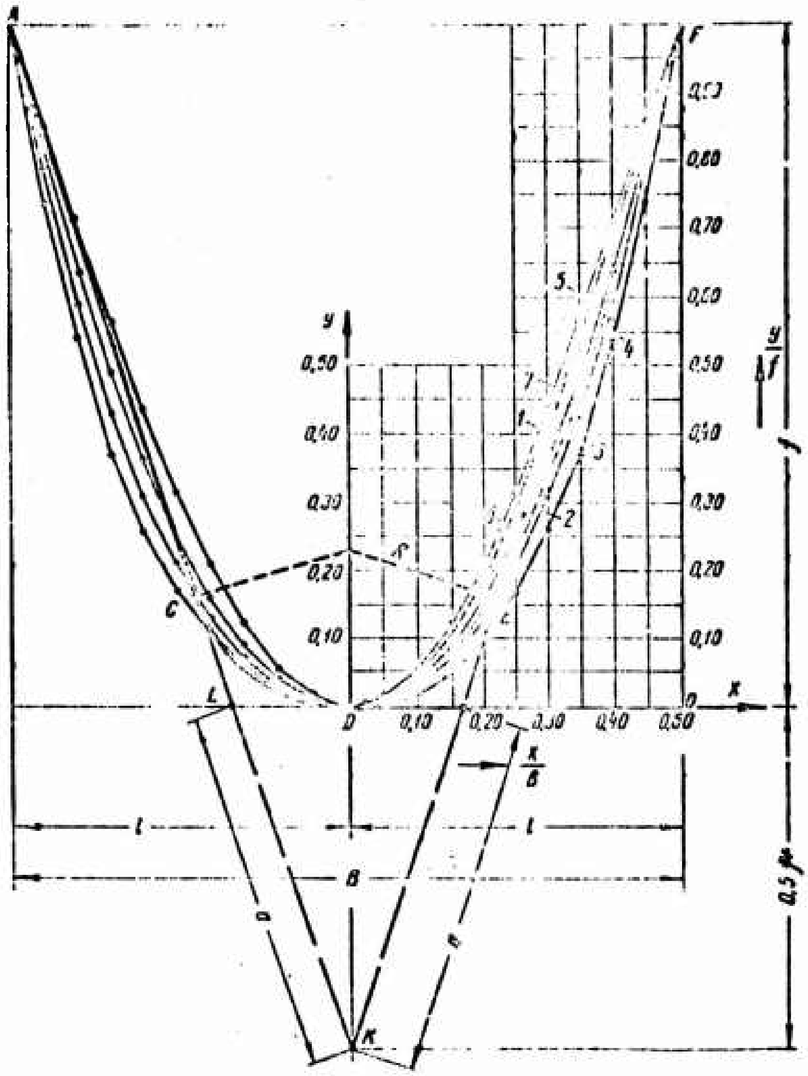

Fig. 303. Funicular curve of the generatrix of the hopper, inscribed in a square.

Let, for example, the vertical component of the load, equal to the half-sum of the weight of the ore and design of the capacitive part, and the angle, composed of the tangent to the curve at this point with plumb-line, at point *A* be known. The value of the horizontal reaction (thrust) and of the resultant loads at point *A* is determined in this instance by means of the usual expansion of forces.

Let curve DF (curve 5) in Fig. 303 extend past the point, whose coordinates, expressed by $\frac{x}{B}$ and $\frac{y}{B}$, are equal to 0.25. Let also the slope of the tangent at this point be equal to 2. Considering the conditions of equilibrium at this point, one ought to apply vertical loads, limited by the vertical lines, passing through this point and through point D. In this way, also here the vertical component of the load is equal to the half-sum of the weight of the ore and of the designs of the capacitive part within the limits of the examined width of the hopper. The horizontal component of the load at this point (thrust) is equal to 0.500, and the resultant loads (forces in the strand) is equal to 1.118 of the value of the vertical component load at this point.

The ore hoppers with a flexible capacitive part, used in the iron-ore industry, are usually characterized by a relatively great height of the parabolic capacitive part consisting in a number of cases, of 0.85-1.00, of its width, which is determined by the relatively large values of the angles of internal friction and by the worst conditions of abrasion, respectively. At a normal profile of the cover of the parabolic ore hopper, inscribed in a square (Fig. 303), sections of the bottom, inclined toward the horizontal at angles, less than 50°, occupy, by design, from 30 to 40% of the entire width B of the cell of the hoppers. With a decrease in the height f and with a ratio of the height to the width of the cover, $\frac{f}{B}$, the given index is made worse. It is recommended to take the value $\frac{f}{B}$ equal to 1.0-0.9; at smaller ratios of these values it is necessary to especially and throughly consider the possible form of the curve of the flexible wire mesh of the hopper.

As is know, with the effect of the arbitrary load of the hopper, rising from zero to a defined amount in the center of the span on a straight line, i.e., with an arbitrary diagram of the load in the form of an isosceles triangle, the equation of the funicular curve, which corresponds to the given load, will have the form

$$y = \frac{f}{2l^2}\left(3x^2 - \frac{x^3}{l}\right),$$

or

$$y = \frac{2f}{B^3}(3Bx^2 - 2x^3),$$

where f — the height of a cell; B or $2\ l$ — the width of the hopper.

The corresponding curve 1 is plotted in Fig. 303 for the cell, inscribed in the square for a value f, equal to B.

Curve 7, obtained as a result of using simplified means of constructing the profile of the hopper, is quite close to the points of the curve 1 in the upper half of the capacitive part. The plotting of curve 7 is understandable from the draft in Fig. 303. The value $R = \frac{f \cdot l}{2(3a - l)}$, where a is the length of segment KL.

One should keep in mind that curve 1 was plotted allowing for the arbitrary delta-shaped load of the hopper; in practice the vertical load changes along the width of the hopper approximately according to the law of the parabola. In this way, curve 1 is plotted in accordance with the relatively crude simplification of the load. Furthermore, in obtaining curve 1 the effect of the horizontal pressures in the hopper, which substantially change the shape of curve has not been completely taken into account.

The parabolic character of the change in the vertical load, and also the forces of the horizontal pressure in the hopper can be taken into account in the graphic construction of the corresponding funicular curve, which should be taken as the line of the bottom of the hopper. There are approximate and accurate means of determining the funicular curve by means of a series of approximations in an analytical form, plotted allowing for the horizontal pressures in the hopper. These methods of determination were confirmed earlier by conducted experiments on a small hopper having a height of 0.87 m and a span of 1 m with loose contents on an angle of internal friction of 40°.

The determination of the funicular curve can also be produced with the aid of a graph (Fig. 304), which consists of loading ordinates at the points of suspension, under the assumption of equality at zero. The curves of a graph correspond to the various values of a magnitude

$$\frac{f}{B}\sqrt{k},$$

where

$$k = \mathrm{tg}^2\left(45 - \frac{\varphi}{2}\right).$$

ϕ – angle of internal friction.

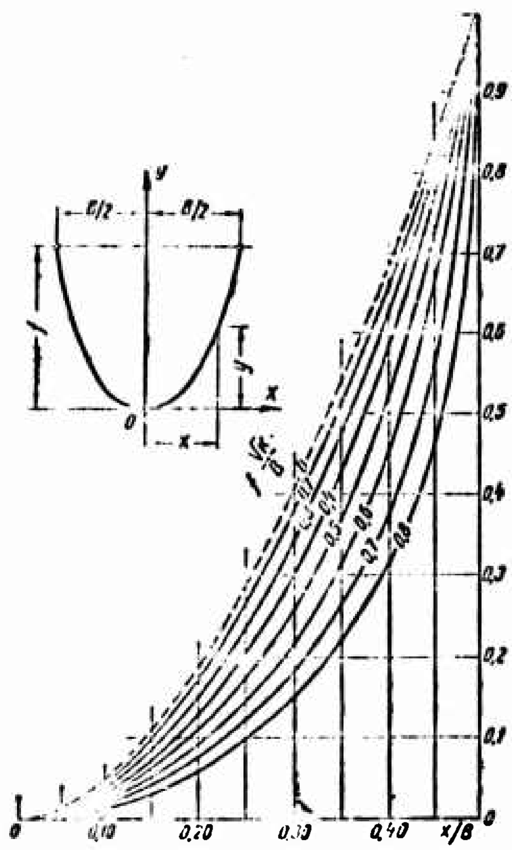

Fig. 304. Graph for the determination of the funicular curve of a suspension hopper.

Among the values ϕ of practical interest there is the average value of the angle of interval friction for iron ore, amounting to 47-49°. In this way, the funicular curve, which corresponds to the angles of internal friction at 45 and 50° deserve the utmost attention.

Curve 2 in Fig. 303 is the funicular curve of the suspension hopper, inscribed in a square, and loaded with loose quicksand material having an angle of internal friction ϕ, equal to 45°. Curve 3 is a funicular curve of the suspension hopper, inscribed in a square, and loaded with loose material having an angle of internal friction ϕ, equal to 38°, which corresponds approximately to crushed limestone and it extends beyond the point of the extreme minimum values of the angle of internal friction for iron ore.

Curve 4, shown in Fig. 303 by a dotted line, is a funicular curve of a suspension hopper, inscribed in a square, and loaded with material with the angle of internal friction of 50°, which corresponds to the greatest (51°30') and most widely used (47-49) values of the angle of internal friction for iron ore.

Curve 6 is plotted under the assumption of a parabolic character of a vertical load, and the horizontal pressure of the loose material here is taken into account. Curve 5 is a conventional square parabola, determined by the equation

$$x^2 = 2py$$

with a parameter p, found from the boundary condition

$$l^2 = 2pf.$$

From the obtained curves, given in Fig. 303, the closest to the actual conditions of a load of the hoppers, are the funicular curve 2 (ϕ = 45°) and predominantly 4 (ϕ = 50°). Curves 2 and 4, especially curve 4, are close to the outlines of curve 5, i.e., parabolas. With an incomplete load of the hoppers, curves 2 and 4, to an even greater degree, are close to the points of curve 5.

The riveting of the rigid frames of a bottom of the discharge openings of the hopper and the rigidity of the wire mesh, the sheets of lining and shockproofing all affect the direction of the reduction in the horizontal displacements of the wire mesh, specifically, they lead to the approach of true curves 2 and 4, from curve 5; true curve 3 from curve 2 and so on. The natural measurements, made on a large suspension hopper with a span of 7 m, height of 6.1 m, loaded with iron ore after crushing, sorting and dry magnetic separation, given here are verified. Figure 305 shows a part of the lower section of this hopper with the funicular curve applied here. As it appears from the figure, the true funicular curve 4 of the loading ore of the hopper and the funicular curve 5 of the empty hopper practically coincide with the square parabola 2.

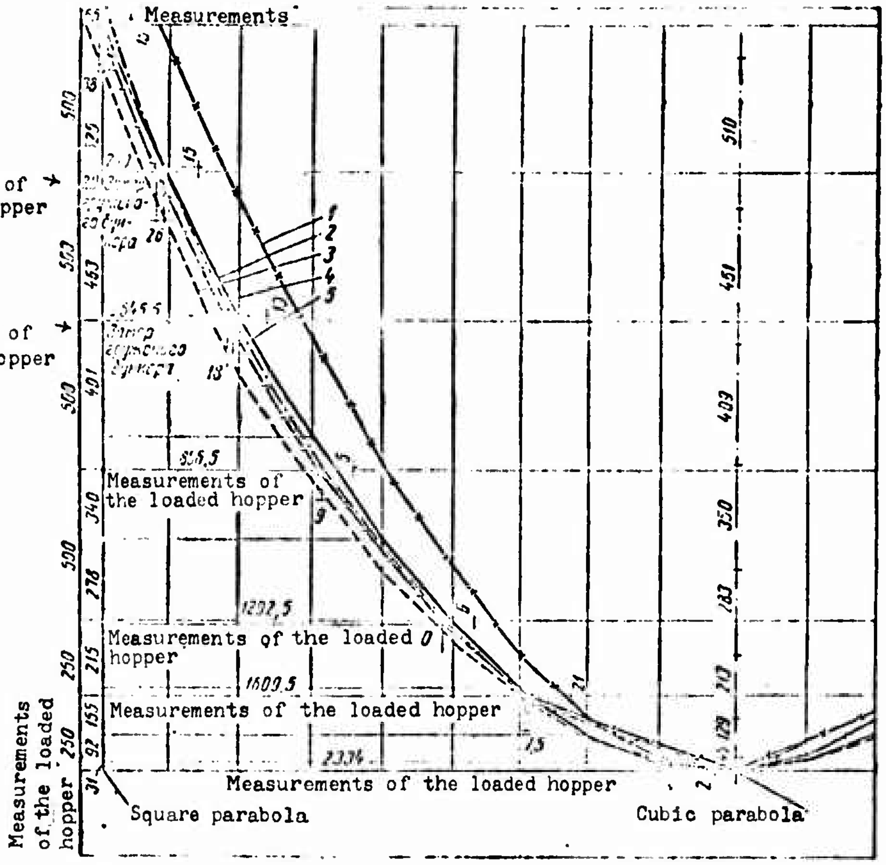

Fig. 305. The results of the natural measurements of funicular curves of a suspension hopper, loaded with iron ore after crushing, sorting and dry magnetic separation: 1 – cubic parabola; 2 – parabola, $x^2 = 2py$; 3 – funicular curve along a projection; 4 – true funicular curve of a loaded hopper; 5 – true funicular curve of an emptied hopper.

On the basis of that proposed, under conditions of storage of ordinary iron ore and the like, based on value ϕ, it is possible to take the outlines of the bottom of the suspension hopper, described for a parabola, determined by the equation

$$y=\frac{f}{l^2}x^2.$$

When $f=B=2l$, just as in the hopper shown in Fig. 303, the equation of the parabola (curve 5) assumes the form

$$y=\frac{2}{l}x^2.$$

With small values of the angle of internal friction ϕ, equal to 35° and less, the outline of the bottom should be determined by means of the construction of a funicular curve, corresponding to the parabolic character of the vertical load and in the presence of horizontal pressure in the hopper. The outlines of the curve can be determined by graphic and analytical means and, specifically, with the aid of the graph shown in Fig. 304. During the selection of the form of the bottom one ought to consider the effect of the various riveted structures on the bottom and the effect of the hardness of the wire mesh, lining and shockproofing. The calculation of these factors can lead to a means of increasing the assigned average value of the angle of internal friction ϕ by approximately 5° in the calculation.